raphy must be accurately mapped, reports must give a careful classification of all timber, viz., species, standing, dead and down, timber suitable for piling, poles and ties, with an estimate of probable defects; also soil classification, conditions of undergrowth and mineral indications.

Topographic surveys for logging operations are made in various ways. The method to be used is determined by the lay of the land and the density of the forest growth. Until a year or two ago the custom was to make all such surveys by the aneroid method, which is simply a refinement of the methods formerly used by timber cruisers. Elevations are determined by primary leveling along certain lines selected as base lines. If the area has been previously covered by the survey of the United States General Land Office, it is customary to establish the base along the section lines; while in unsurveyed territory irregular traverses are run along roads, trails or ridges, the work usually being done by transit and stadia and the elevations computed at fixed intervals. From these base lines parallel "strips" are run.

The strip crew usually consists of a compassman and an estimator. The compassman runs a rough line with a box compass and paces the distances. He carries an aneroid barometer which he sets each morning from the reading of a camp barograph. He sketches the contours each side of the line run. It is necessary to record the time at which the aneroid readings are taken in order to make the corrections for the fluctuations of the instrument during the day.

The aneroid, while a handy instrument for taking rough elevations, is too eccentric to be relied on to do accurate work (as any engineer is aware who has used it in mountainous regions subject to frequent and sudden atmospheric changes). As the working aneroid goes through the changing atmosphere conditions during the day, it is necessary to correct the readings in proportion to the fluctuations of the camp barograph.

After the day's fieldwork is completed and the aneroid readings corrected, the difficult work of adjusting contours

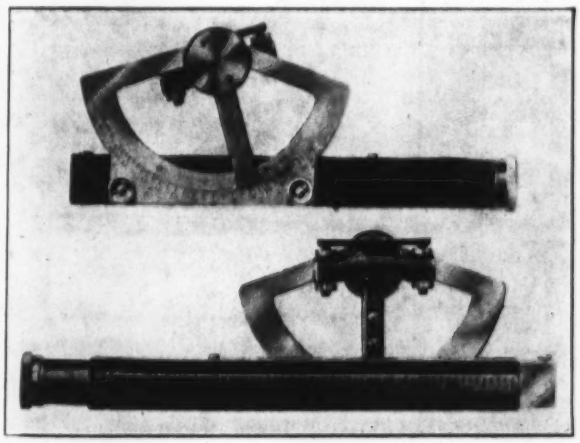


FIG. 1. IMPROVED ABNEY LEVEL OR CLINOMETER

on the finished map still remains. If the topographer has been conscientious and is skilled in his work, he has gathered a wealth of topographic detail which is all to be lost and wasted when his contours are put through the "juggling" process. The very lack of preciseness and the knowledge that the map must be adjusted greatly discourages careful and conscientious sketching in the field.

It is very evident that the map cannot be accurate. All topography is approximately correct, but it would hardly answer as a map on which to make the paper location of a railroad or a flume. The cost of making surveys using the method outlined is from 5c. to 35c. per acre.

Such was the condition when the engineers of District No. 1 of the United States Forest Service commenced their series of experiments to develop a more efficient method of topographic surveying. Ordinary topographic-survey practice was not applicable. Heavy timber and dense

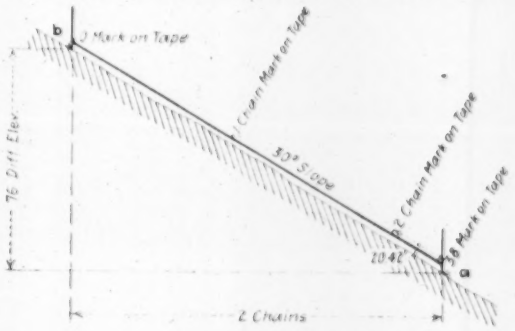


FIG. 2. ILLUSTRATING USE OF SPECIAL SLOPE TAPE

brush and undergrowth prevented efficient use of the plane-table method, as it was necessary to get beneath the cover of the forest in order to get the topographic detail required. Also the operations must necessarily be combined and coördinated with the timber estimating and appraisal work and could not therefore follow the methods that would be adopted for the production of a topographic map alone. Low cost was an important consideration.

The first tests were made with the ordinary Abney level, or clinometer, familiar to all engineers. While the results were satisfactory, it was a cumbersome method, in which the field men were compelled to constantly refer to reduction tables in order to ascertain their true horizontal position. The instruments as then made were not constructed for such work, and it was almost impossible to keep them in adjustment. By constantly experimenting and making improvements a new Abney level was developed that is now manufactured [by Keuffel & Esser, New York] expressly for work of this character. Simplified methods introducing greater speed and accuracy were worked out.

New Abney Level and Its Use

The improved Abney level, illustrated in Fig. 1, is only slightly larger than the old Abney, and its cost is about the same. The arc is made larger and graduated to read directly the difference in elevation per chain (66 ft.) of horizontal distance. A prism has been substituted for the german silver reflector, and the bubble tube is made longer, with adjustment devices similar to those on the bubble tube of a transit. The bubble-tube support is attached rigidly to the indicator arm of the arc, thus eliminating the inevitable lost motion between these two parts existing in all old-style Abneys.

The bubble tube is arranged to allow adjustment to eliminate refraction, which made the old-style instruments so unreliable in measuring steep slopes. A semicircular lens placed in the fore end of the eye-piece tube, as in the Locke level, magnifies the movement of the bubble so as to greatly facilitate accurate sighting. The various

changes introduced have so improved the instrument and the results of its intelligent operation have proved so surprising that the new Abney will doubtless meet with favor among the entire engineering profession, and its use in fieldwork of all kinds will probably be greatly extended.

Aside from the special graduation shown in Fig. 1, which is applicable to all topographic work, plates with the usual degree and per cent. graduations are furnished, and also a special graduation to give directly horizontal distance from slope chaining. This graduation has found much popularity in cadastral surveys.

Topographic Surveys of Timber Lands

There was considerable objection at first to the Abney method on account of the necessity of taping the slope distances, but two years of trial have proved that this work can be done with practically the same convenience and time as pacing.

The unit of measurement adopted was the chain of 66 ft., for the reason that a large part of the work is the retracement of land-survey lines recorded in that unit.

FIG. 3. LOCATING CONTOURS BY THE ABNEY METHOD

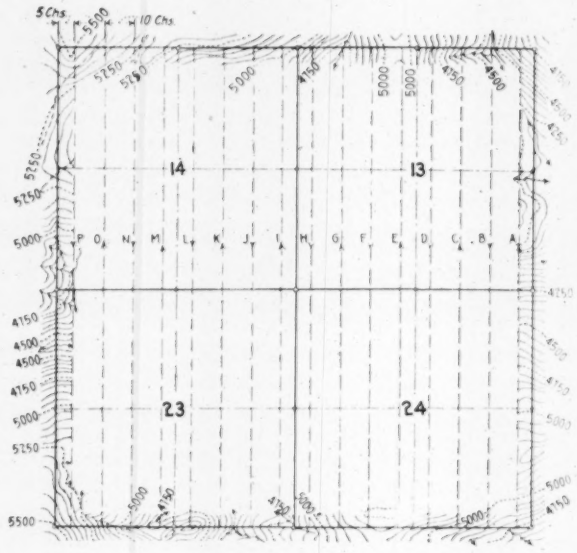
and also for convenience in timber estimating. Obviously a definite horizontal unit is necessary for operation without the use of tables and after thorough trial it was found that a distance of 2 chains was most convenient for rough country covered with heavy timber. A special 2½-chain tape has therefore been designed for use with the Abney method, the tape proper consisting of 2 chain lengths divided into links, with an additional half-chain "trailer." The trailer is graduated in order to allow the proper slope measurement for the excess in length of the hypotenuse of a triangle over a base which is 2 chains.

For example, suppose the crew to be ascending a slope, as indicated in Fig. 2. The topographer reads from the arc that the rise per chain on that slope is 38 ft. The rear chainman (usually the timber estimator) then looks for the 38 etched beyond the 2-chain tag on the tape, which in this case would be 20.42 ft. beyond. By stretching the tape tight, the point *b* is then located and is known to be 2 chains distant from, and 2x38 ft. higher than, *a*. The topographer then sketches his contours, taking sideshots in order to locate the distance between contours and "sidewalking" if necessary.

A strip crew of two men will cover about two miles of line a day. Frequently the brush and undergrowth are so thick that sights two chains long are impossible, requiring the use of a one-chain distance or sighting by sound. Remarkably good closures have been obtained in many cases employing the latter method. In dense and dark forests it is usually necessary for the estimator to carry a small mirror in order that the topographer may sight on the flash. In this way sights are obtained through brush that seems almost impenetrable to the eye.

The general method followed is similar to that used in the aneroid method. Primary control fixing the geographic position of the area both horizontally and vertically is obtained by a precise connection to adjacent primary stations of the United States Geological Survey. Secondary control lines are run around a block of sections, or topographic unit, stakes or posters on trees being left at periodic intervals, from which the strip surveys may be begun or closed.

The Abney instrument is used for running the control lines as well as for the strips. It has been found to be about as accurate as direct leveling and much faster. The



FIGS. 4 AND 5. STEPS IN THE TOPOGRAPHIC SURVEY OF FOREST LAND BY THE ABNEY METHOD

work on control lines is of course much more carefully executed than on the strip lines, two Abneys being used.

The control system is so planned that the strip surveys will not be run more than two miles without closing upon a control line. The strip surveys are run along parallel lines usually ⅛ or ¼ mi. apart, depending upon the density of the timber and the degree of detail required.

It is customary to combine the mapping with either the timber-estimating or the soil-classification work, thus making one survey serve two purposes and reducing the

cost. The crew consists of the topographer, who keeps his direction with a staff compass, operates the Abney and sketches the topography; and the estimator, who "snubs" the rear end of the tape, making the proper allowance for slope, and estimates the timber on a strip 33 ft. wide on each side of the surveyed line. Experience has shown that the time lost by the estimator in holding the rear end of the tape is practically negligible.

The strips are usually 10 chains wide, the line being run through the center of the strip, and the topography sketched for 5 chains on either side. The method is not a system of determining the elevation of points along a profile line and interpolating the intermediate contours, such as the aneroid method is, but a number of points are located by taking side shots at right angles to the general direction of the contours, as indicated by the dotted lines and arrows on Fig. 3. Although the skeleton of the topography is thus accurately obtained mechanically, the sketching or filling in of the topographic detail depends entirely upon the skill and judgment of the topographer, which may be attained only through considerable experience and some amount of natural talent, often called "topographic sense."

As the price of stumpage increases, the accuracy and detail demanded on topographic maps increase. Some of the larger lumber companies now have their strips run at 2½-chain intervals, estimating each tree separately and trusting nothing to averages.

Accuracy of Abney Method—Errors of Closure

The results obtained by those who have been working with the Abney method outlined are truly marvelous. On one project comprising 54,000 acres in the Cœur d'Alene National Forest in northern Idaho, completed during the 1915 season, 64 mi. of control line was run with an average closure in elevation of one foot to the mile.

The area mapped was very rough and mountainous, and the timber cover consisted largely of heavy stands of Idaho white pine accompanied by heavy undergrowth and dense brush. The greater part of the area had been previously covered by both the General Land Office surveys and by the Geological Survey small-scale topographic quadrangles, so that no primary control had to be executed. On this same project over 500 mi. of strip surveys were run with an average closure in elevation of 10 ft. to the mile. The minimum error on such lines was one foot to the mile, while the maximum allowed was 30 ft.

The errors of closure for alignment and distance on the strip surveys each averaged ½ chain to the mile. Mapping was done on a 4 in. = 1 mi. scale, using a 50-ft. contour interval. It is readily apparent, therefore, that the errors of closure resulting can be readily adjusted so as to be almost negligible.

The cost of the work, including the control, mapping, timber estimating, etc., was approximately 12c. an acre.

FIGS. 6 AND 7. TYPICAL IDAHO FOREST SURVEYED BY ABNEY METHOD

The results of this project furnish an average example of what is being accomplished on a large amount in similar work. In less rugged and more open timber, as in the yellow- and lodgepole-pine stands, for example, closing errors and costs are more favorable.

Testing Concrete in Roads—The United States Office of Public Roads and Rural Engineering has begun a series of tests at Arlington, Va., to determine the proper thickness of the concrete base for roads of different types when subjected to loads of different magnitudes and when placed on various kinds of sub-base. In view of the increasing intensity of wheel loads to which roads are now being subjected, the solution of this problem is becoming of great economic importance. Steps have been taken to mold slabs of varying thicknesses on several kinds of sub-base and test them to destruction, the load being applied through various kinds of paving surface. Concrete roads are now being built throughout the country with numerous kinds of aggregate and under various construction methods. There is a noticeable difference in the amount of wear suffered by these surfaces, and it is a much-mooted question as to which kind of aggregate will prove most economical under particular conditions of traffic and location of source of supply. In order to gain information of value in this connection, an instrument has been devised and constructed in the concrete laboratory at Arlington, Va., for the purpose of measuring the amount of wear of concrete roads of different types. Periodical wear measurements will be taken of concrete-road surfaces on which the traffic conditions are known. It has long been known that concrete is not elastic, but attains permanent deformation under working loads. During the course of a series of tests on reinforced-concrete slabs recently made by the Office of Public Roads and Rural Engineering, and now being continued, it was very forcibly brought to the notice of the observers that the concrete continued to deform slowly as long as the load was applied. This fact is significant, as it helps to explain certain stress conditions in concrete roads and bridges when subjected to temperature changes. In order to study this phenomenon more fully, a series of tests has been begun to determine the "flow" of concrete and reinforced concrete when subjected to direct compression and to bending stresses. Working conditions will be maintained on the specimens as long as they exhibit "flow," and finally the specimens will be broken in comparison with like specimens that have not been previously loaded. From the Departmental Circular of the United States Department of Agriculture.

Field and Office

Cleaning Hollow Piles With a 10-In. Orangepeel

By G. L. Knight*

In building a heavy steel platform foundation for turbine No. 3 in the Gold St. generating station of the Edison Electric Illuminating Co. of Brooklyn, N. Y., it was necessary to put down a number of 12-in. tubular-steel piles, which later were filled with concrete. The piles were dredged out by a method employed only infrequently—by the use of 10-in. orangepeel buckets. The piles could not be blown out with air, owing to a buried timber cribwork dock and many boulders through which the piles were driven. The use of water had the added objection of possibly undermining the adjoining foundation of No. 2 turbine. The work was done in cramped quarters. Before driving could commence, an existing mass-concrete foundation, which occupied the site, had to be cut out.

The piles used were cylindrical sectional piles of 12-in. inside diameter, driven in sections of convenient length, the joint being made by means of a sleeve 12 in. long and of the same outside diameter as the inside of the pipe. The sleeves were held in place by centerpunching the pile sections 6 in. on each side of the joint. The material was excavated from inside as the pile was driven. The first sections used were 18 ft.

On account of the proximity of No. 2 foundation, which was not piled, but whose bottom rested directly on the soil, 12 ft. below the basement floor, it was not deemed advisable or safe to jet the piles. The method adopted was to drive as far as possible and then excavate with a 10-in. Hayward orangepeel bucket inside the 12-in. steel pile.

The piles drove anywhere from 5 to 10 ft. in the first driving. The instructions given the inspectors were to drive until the pile came to refusal (¼ in. in three 5-ft. drops of the hammer) or until they heard the ring of a boulder or got the spring of timber.

At the end of the first driving the clearing crew started in, using an auger, a chisel bar and the bucket. The auger proved of little use as there were too many small stones and other obstructions to permit its turning, although it was used whenever the chisel bar showed that there was a section of sand or soil fairly free from stone. The procedure throughout the work after each driving was to use the chisel bar to loosen up the material inside the pile and then with the bucket remove the material to within a foot of the bottom of the pile.

FIG. 1. THE DWARF BUCKET USED TO DREDGE OUT PILES, WITH SAMPLES OF THE SPOIL

Piles Encountered Many Obstructions

Over part of the site of this foundation there had apparently been an old crib dock or wharf whose bottom rested some 25 ft. below datum (basement floor is datum), and this accounted for the many obstructions encountered. These were principally boulders, rough stone and the timber itself, many pieces of which lay horizontally in the path of the piles and were cut through and a section brought up by the bucket. Fig. 2 shows a few typical prizes recovered.

As is usually the case with such jobs, the progress was very slow and unsatisfactory at the start of the work, but as soon as the crews got used to their duties the

*Designing Engineer, Brooklyn Edison Co., 360 Pearl St., Brooklyn, N. Y.

work went on with remarkable speed considering the difficulties encountered. There were two driving crews (one day and one night), and three excavating crews (two day and one night). Three buckets were used, so as to be sure to have one spare at all times.

At the north end of the excavation a section of concrete was encountered, which had once been used as a sump and which lay directly in the path of the three piles at the north end. As the top of this concrete was some 12 ft. below the surface, it was impracticable to excavate it in the open, and it was determined to drill three holes in the concrete to admit of the piles being placed in their proper positions. A well drill was secured and placed in the ways of the piledriver, and the three holes—15 in. in diameter and about 5 ft. deep—were cut through in four days. The three piles were then inserted in the holes and driven to place.

The depth set for the bottom of the piles, after careful examination of the soil excavated, was 10 ft. below the bottom of the condensing tunnel, or 34 ft. below datum.

In some cases where the pile encountered a boulder but glanced off, notably in the case of piles 8 and 11, the bucket was blocked after one driving, although the driving indicated nothing unusual. Where the pile had been driven only a few feet, it was pulled up and replaced with a slight change in location, the work proceeding then very cautiously, using a smaller core drill inside the pile to break through the boulder in case it was hit again. Not only were the bottoms of the piles damaged, but in some cases the second section was telescoped with the first. In another case the sleeve slipped and the upper, or head, section was driven eccentric to the bottom section for a distance of some 2 ft. before it was discovered.

As the piles were laid out as close together as practicable, it was not possible to drive other piles to make up for the loss of these. Each case had to receive careful consideration on its merits. Sometimes the pile was pulled, and where this was impossible it was filled up and the piles adjacent driven to greater depth. In still other cases, where the break was near the surface, a sheeted pit was made around the pile, the material excavated in the open until the joint was uncovered, the damaged steel cut off and a new section put on. In all, 23 piles were driven in one month, working 24 hr. per day. Subsequently, load tests of 40 tons were made on several of the piles.

Erecting a Bascule Bridge

Demands of navigation quite frequently make it necessary to erect bascule bridges in an open position. This requirement brings up special problems in so locating and adjusting the structure that it will operate smoothly and have even bearing when closed. The following notes give some of these problems and their solution in the case of a bascule span of about 175 ft. clear waterway.

The girders were fabricated in three sections, *A*, *B* and *C* of Fig. 1, with field splices along lines *X Y* and *Y Z*. The first steel erected was the rack and its support, which provided means for staying the rocker section *B* until the temporary bottom lateral system could be placed. The rocker section was placed in the extreme open position for the span and held there by the erection strut, which engaged one of the slots in the tread at its top end and was wedged against a tooth of the track plate at the bottom (Fig. 2).

It was found that in fabricating the sections *A* and *B*, which have a heavy flange along one side and nothing but the web plate at the other, the riveting of the flange had caused that edge of the section and web plate to stretch and the other, unstiffened, edge to buckle. When section *A* was put in place, it was found that the long splice *X Y* came together at the ends while the plates were still considerably separated at the center. Yokes and clamps were of very little assistance in getting the splice together, the most effective method being to use a dozen or more drift-pins in the splice plates at one time. In one case it was impossible to get fair holes by ¼ in., and the $\frac{13}{16}$-in. holes were therefore reamed to $1\frac{1}{16}$ in. and 1-in. rivets driven.

To align the girders so that the rolling segment would be parallel to the bridge center line, a transit was set

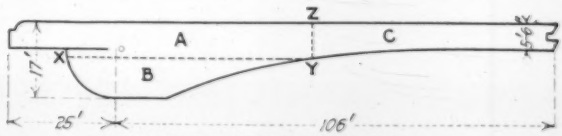


FIG. 1 CANTILEVER GIRDER FOR ONE LEAF

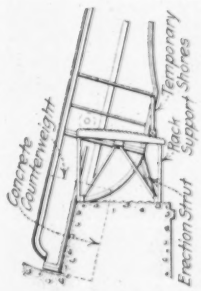


FIG. 2

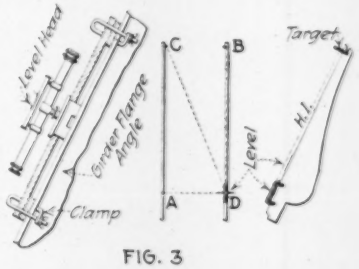


FIG. 3

FIGS. 1 TO 3. DETAILS OF BASCULE LEAF BRIDGE

Fig. 1—Bascule girder made in three erection pieces. Fig. 2—Heel portion of bascule leaf as erected. Fig. 3—Alignment sighting to detect twist of girders

up on the track plate in a position offset from the girder center line because of the erection strut's interference. A foresight with the same offset fixed the plane parallel to which the rolling segment was to be adjusted. By taking a position with the transit on the girder center line across the channel, it was determined how much the tops of the girders were away from the vertical. The necessary adjustments to bring the tops to correct position were made by tightening with steamboat ratchets the temporary wire-rope diagonals of the lateral system. To set the girders so that they would be in the same position of roll, a steel rod gage or standard was used, with which the distance from the center of bearing (closed position) on the track plate to the same point on the rolling segment could be measured. Adjustment could be had by the wedges at the foot of the erection strut.

Certain blocking against the counterweight box was inadvertently removed just before concreting was commenced. The result of this mishap was to allow one girder to roll back to a more nearly erect position, enough to lift it off the erection strut ⅛ in. When a check was made to find the effect, it was seen that one girder was ahead of the other at the top, which was determined by the following method:

The legs were removed from a Y-level, which was then mounted on a board (Fig. 3). This board was clamped to the cover plates of the girders so that the axis of ro-

tation of the telescope was normal to the bridge floor. The H.I. was taken as the distance from the back of the flange angles to the line of sight of the telescope. The telescope was then adjusted by the leveling screws until it was made to revolve in a plane parallel to the three points *D*, *B* and *A* (Fig. 3) at a distance equal to the H.I. above them. A reading was then taken to the back of the flange angle at *C*, which gave the error in its position. Self-reading targets were used at *C* and *B*, bolted to the girders.

The concrete had set by the time the error was discovered, and it held the two girders firmly in position relative to each other. It was necessary that the outer ends of the girders be in correct position, or else the bridge would not lock properly when closed. As no adjustment was possible at the lower end, the upper ends had to be rotated slightly about the splice *ZY* (Fig. 1), the holes in which were then reamed. Half of the error was taken up in each girder.

On the opposite side of the river the only trouble encountered was in the position of the track girders. They were carried by buttresses that were concreted to a point slightly below the position of the bottom flange of the track girders before the latter were placed. The track plates were set accurately to level and to line and station. The concreting was then finished. Afterward it was found that the pressure of the concrete had in some way become unbalanced on the two sides of the girders, and one of them had moved, making the distance between the track plates too great by ½ in. In this case it was possible to take up part of the discrepancy in the setting of the rolling girders and part by increasing the spacing of the girders, but the accident suggested the desirability of providing temporary sway-bracing between track girders in such cases.

Occasions for the use of level or transit with legs removed, as in the case above, come up very frequently in the engineering work on bridge erection, and the instrument man should remember that the legs are by no means an essential part of his apparatus. Many times the use of the head alone will make possible a set-up which would otherwise be difficult or impracticable.

Overdriven Piles and Poor Subaqueous Concrete

By R. W. Stewart*

The bed of the Los Angeles River is composed of sand and gravel nearly free from clay or silt. On account of the excellence of this material large quantities are hauled out for construction work. The rate at which the gravel is replenished by floods carrying down a new supply from above is much slower than the rate of removal. As a result the channel through the city has lowered 10 to 20 ft. and it has become necessary to underpin or reconstruct the piers of all the older bridges. This work has uncovered some engineering data of interest.

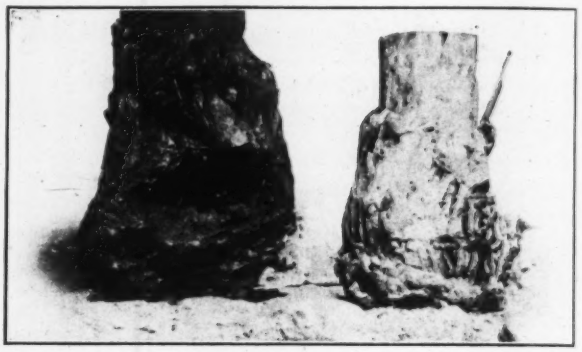
Several of the bridges at sites where groundwater interfered with excavation to suitable depth had pile foundations under concrete piers. It was specified in all cases that these piles be driven to refusal, and evidently there was no laxity in enforcing the specification.

*Engineer of Bridges, City of Los Angeles, Calif.

Of 280 piles which have been excavated, 130, or 47%, were either broomed or telescoped in driving. By a broomed pile is meant one which is split and expanded into a mass of fibers at the bottom, thus causing it to resemble a broom. Some of the telescoped piles had a neatly formed telescopic joint which resembled a pipe coupling. The view herewith shows the lower ends of some broomed piles.

An experienced piledriver foreman can tell when a pile becomes broomed in driving by observing that it rises slightly immediately after the blow of the hammer. It is probably needless to say that a broomed pile will settle under load and is of little value. Care should be exercised to stop driving before rupture of the pile begins.

The concrete in the lower portions of some of the piers had been deposited under water. This concrete was in



LOWER ENDS OF TWO BROOMED PILES

such condition that a man wielding a piece of wooden two-by-four by hand could punch it to pieces. The fact was well illustrated that in depositing concrete under water it is necessary to use a device to prevent free concrete from sinking through the water.

In reconstructing these piers it was desired entirely to avoid placing concrete in water. Difficulty was encountered, due to the streams which would flow in through the sides of the foundation pits and surge up vertically at points in the floor of the pit. The only satisfactory manner of placing the concrete was found to be by building a canvas boat which was enough smaller than the interior of the pit sheathing to allow clearance for the flow of water and the suction pipes of the pumps around it. The water was then pumped down and the boat, while resting on the bottom, was filled with concrete.

Practice in Sewer Construction at St. Paul, Minn., is described in the recently issued annual report of the Commissioner of Public Works as follows: All sewers of 36-in. diameter and under have been constructed of vitrified clay pipe. For large sewers in open-cut work the practice has been adopted of giving the contractor the option of building the sewers of monolith reinforced concrete or of vitrified segmental sewer blocks. The contract plans are drawn showing the details of construction to be used in either case. The specifications require that in wet or soft ground the segmental blocks shall have a concrete backing and in such cases as have occurred, where these conditions were met with, it has proved more economical to use the reinforced-concrete type of construction. All concrete sewers are provided with an invert of paving brick. Segmental block sewers have been built ranging in size from 42 to 60-in. diameter, in several cases, where a dry and firm foundation rendered any concrete backing unnecessary. In all cases the blocks are required to be bedded in cement mortar to insure a firm bearing and to guard against the danger of leaky joints. No difficulty has been experienced with this type of construction and the resulting sewers are entirely satisfactory.

New Features in Concrete Spouting Plant

The new Field Museum of Natural History, in Grant Park, Chicago, will be an imposing marble structure carried on massive concrete piers with pile foundations. It will be 706 ft. in length, with a width of about 450 ft. for the central portion and 335 ft. for the end portions. The central portion will be about 75 ft. high from roof to ceiling, and the side portions will be divided into bays alternately one and three stories high, thus affording ample light for the comparatively narrow upper floors.

The structural design provides for a series of large monolithic concrete piers, which support steel framing for the reinforced-concrete floors. There will be no interior columns, the floors having long spans between the piers and the latter serving to divide up the interior space into separate but connected rooms. In order that the entire structure may be carried by these piers and thus be independent of any settlement of the surrounding ground, the exterior piers have cantilever brackets, which will support the floors and retaining walls of the areaways and entrances. Some of these piers and brackets are shown at

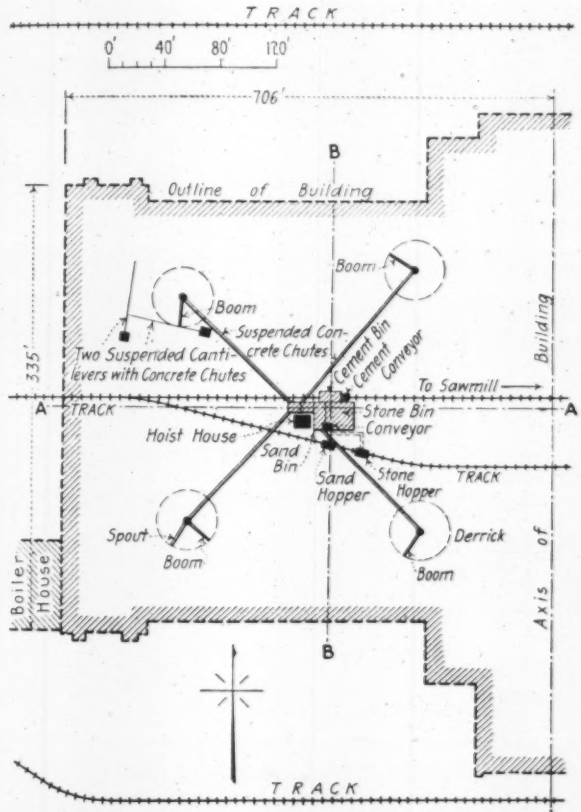


FIG. 1. PLAN OF CONSTRUCTION PLANT FOR THE FIELD MUSEUM, CHICAGO

the left in Fig. 4, which is a general view of the work (taken Nov. 5).

The building is on made ground not yet filled to final elevation, and a special feature of the work is that the substructure, or basement, is being built above the present level of the ground. It will be covered later when this part of the park is filled and graded around the museum.

The site is land reclaimed by filling in along the lake front, a timber and crib bulkhead forming the outer or water line of the park. The depth of water was from 10 to 15 ft., and the present elevation of the site is about 9 ft. above water line. The elevations of the new building will be 34 ft. above water for the basement, 40 ft. for the terrace surrounding the museum and 50 ft. for the first floor.

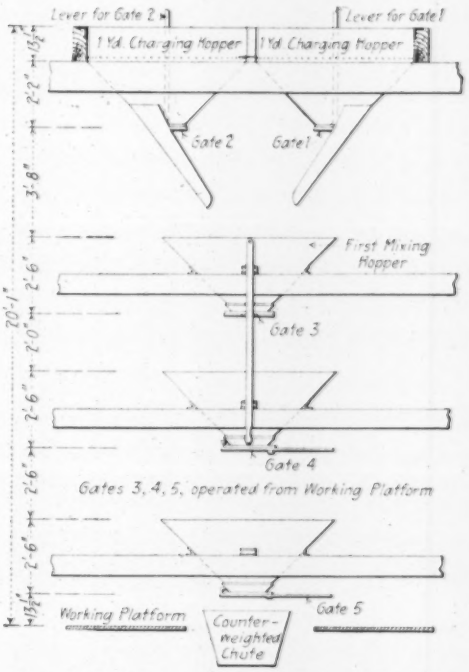


FIG. 2. SECTIONAL ELEVATION OF GRAVITY CONCRETE MIXER WITH DOUBLE CHARGING HOPPER

The entire area is underlain by the Chicago clay, which is mainly a stiff blue clay (with some softer streaks and pockets), changing to hardpan in the lower strata. The foundations are 60-ft. timber piles, grouped under the various footings and cut off at 21 in. below water level. They are driven by steam pile-hammers, long followers being used to put them down to the required depth.

The pier footings are staked out on the surface, and sheeting of single planks is driven to inclose the spaces. The ground is then excavated to and around the piles, the sheeting being braced as the excavation proceeds. The forms for the concrete are then placed. The base of the footings is 30 in. below the water level, or 9 in. below the pile heads.

Details of Concrete Mixing Plant

The construction of the substructure will be carried out in two sections, the west half being built first. The concreting plant is placed at the center of the west half, so that its spouting system can cover the entire area of this section of the work. This arrangement is shown by the plan, Fig. 3. Four railway tracks run across the site and are connected with the adjacent main line of the Illinois Central R.R.

Cement is delivered in bulk in box-cars and is plowed out of the cars by a man handling a scraper, which is operated by a cable and drum. The cement falls directly into the boot of an elevator. Sand and stone are deliv-

ered in drop-bottom cars and dumped at two track hoppers, each of which feeds a short lateral belt conveyor delivering the material to an elevating conveyor. The three materials are discharged into elevated storage bins having capacity for about 400 bbl. of cement, 200 cu.yd. of stone and 100 cu.yd. of sand. From these bins the materials are fed through gates to the measuring hoppers of the mixer.

The concrete is made in a Hains gravity mixer, consisting of three superimposed hoppers, as shown in Figs. 2 and 3. To expedite the work, this mixer is served by two charging hoppers, each of 1-yd. capacity and employed alternately. A man at the top operates the gates of the charging hoppers, and a man on the working platform below operates the gates of the three mixing hoppers in succession. From the last hopper the concrete flows through a chute to a 1-yd. elevator bucket. With the two charging hoppers the concrete can be mixed at the rate of 100 batches (or 100 cu.yd.) per hr.

The elevator tower is 8x16 ft., 175 ft. high, and the bucket is hoisted at a speed of about 300 ft. per min. by a Thomas electric hoist with a 100-hp. motor. The bucket dumps automatically (by tilting) into a hopper which can be shifted to any desired height in the tower. This hopper is of 2-yd. capacity, so that it can hold two batches if the distribution is temporarily checked. It has a four-way gate serving four lines of spouting. This gate is operated by a man at the hopper, his working platform being housed as a protection against the weather.

The system of distributing the concrete by gravity spouting is of special interest on account of its flexibility and wide range of reach, enabling practically all the concrete to be spouted directly into the various forms with very little use of wheelbarrows or hand carts. The entire distribution system, including the tower bucket and hopper, was supplied by the Insley Manufacturing Co., and is shown in Figs. 2, 3 and 5.

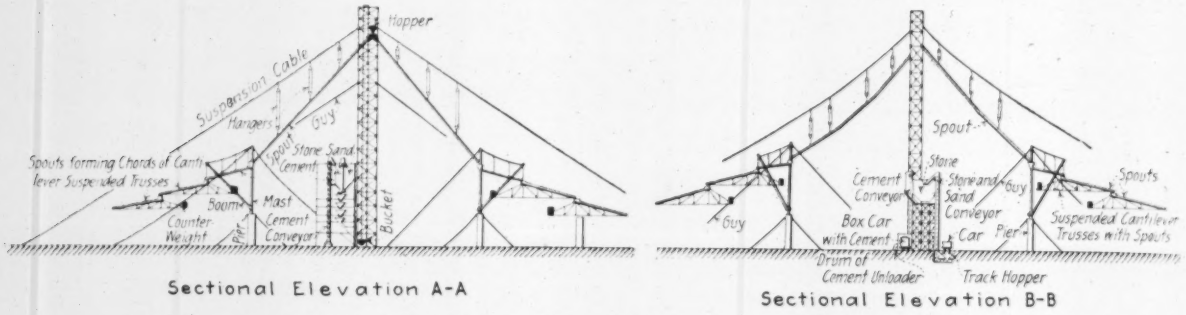


FIG. 3. CONCRETE-MIXING AND DISTRIBUTING PLANT FOR THE FIELD MUSEUM, CHICAGO

The tower is placed at the center of the site, Fig. 2, and from it extend four lines of steel chutes or spouts. These are supported by hanger tackles from anchored suspension cables, as shown. The hangers enable the height and slope of the spouts to be adjusted as required, the free

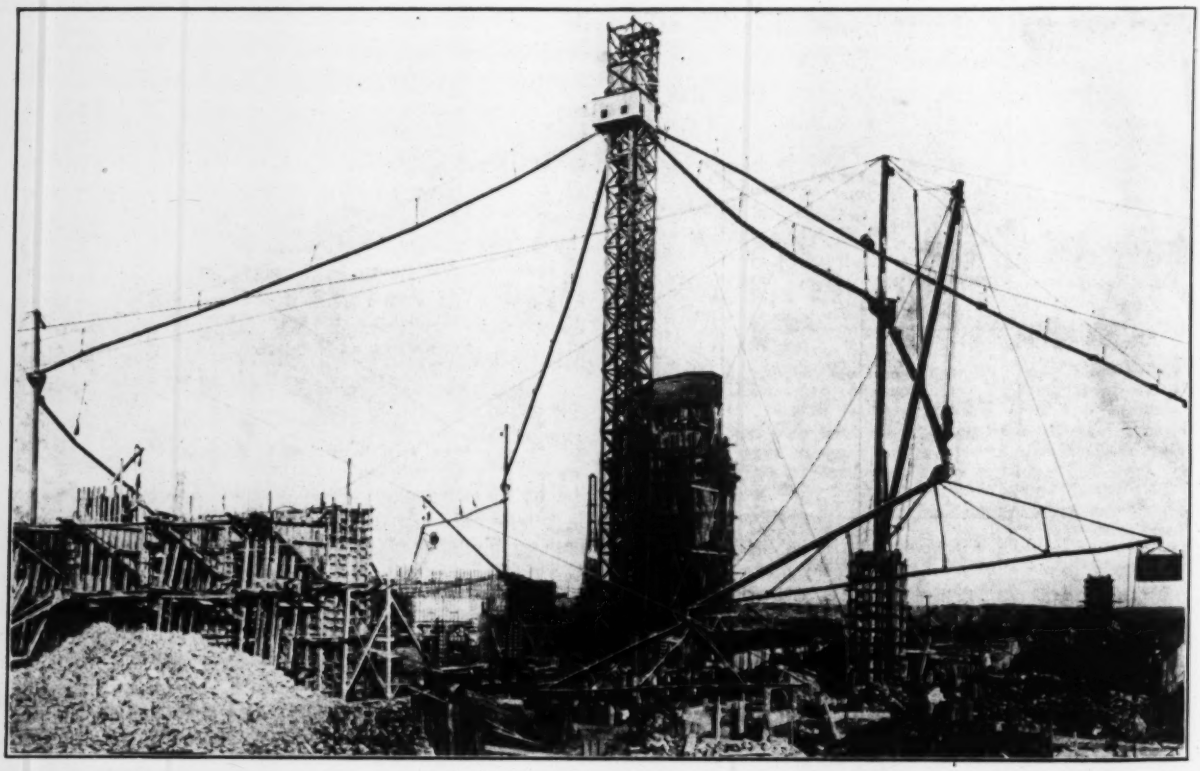


FIG. 4. CONCRETE-MIXING AND DISTRIBUTING PLANT FOR THE FIELD MUSEUM, CHICAGO

ends of the ropes being led to convenient points on the ground. The spouts are 150 ft. long, and each serves an articulated system of spouting, which can be swung in all directions.

At the end of each 150-ft. fixed spout is a guyed derrick mounted on a timber tower or (later) on one of the concrete piers of the building, about 30 ft. above the ground. Each derrick has a 60-ft. mast and 50-ft. boom and carries a spout 40 ft. long suspended by hanger tackles

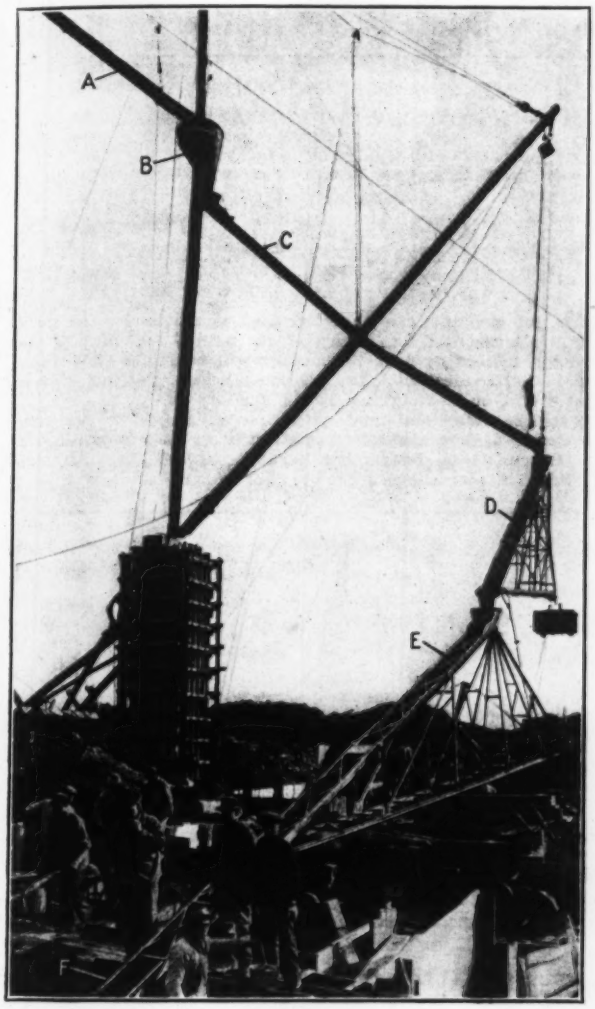


FIG. 5. FLEXIBLE SPOUTING SYSTEM FOR DISTRIBUTING CONCRETE

A—Spout from tower. B—Hopper sliding on derrick mast. C—Spout swung by derrick. D and E—Suspended cantilever trusses carrying spouts. F—Portable spout leading the concrete into the form

from the topping lift. The upper end of the spout has a conical hopper with side discharge; this slides on the mast (the mast passing through the hopper).

From the boom is hung a triangular cantilever truss 60 ft. long and 12 ft. deep. On the top chord of the longer arm is a 40-ft. length of spouting, while the end of the shorter arm carries a concrete counterweight. From the end of this truss is suspended a second and similar truss. This arrangement makes three 40-ft. lengths of spouting, with articulated connections so that each can be pivoted around the end of the one above.

With the swinging of the derrick boom and the trusses practically all points within a wide area can be covered. The derricks and main lines of spouting are shifted as required. Fig. 1 gives a general view of the mixing and distributing plant. Fig. 5 shows in more detail the flexible portion, which comprises the derrick chute and two swinging cantilever chutes.

The architects are Graham, Burnham & Co. The Thompson-Starrett Co. has the general contract, and Robert W. Grange, Jr., vice-president and Western manager, is in charge of the work.

Test Comparing Steam and Drop Hammer Pile Formulas

In the foundation work for the Pearl Harbor dry dock at Hawaii, Civil Engineer E. R. Gayler, U. S. N., had nine piles driven with a drop hammer and eleven piles with a steam hammer in an area having soil of uniform consistency and therefore presumably of uniform bearing value. From the data so obtained he made the following comparison of the accepted bearing-value formulas.

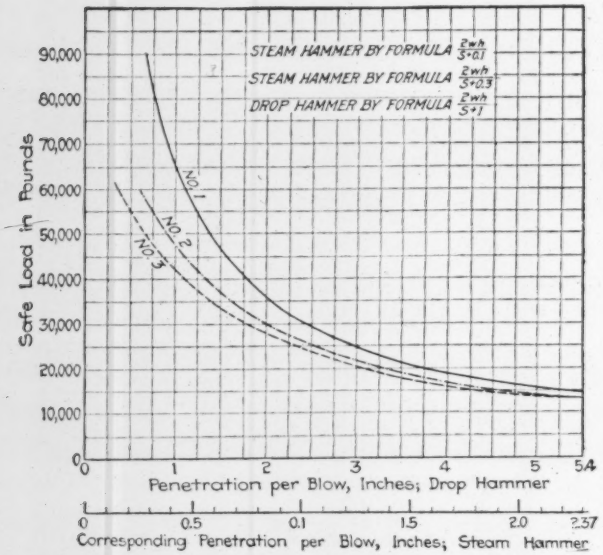
The drop hammer weighed 3,460 lb., and the fall was uniformly 12 ft. The steam hammer had a moving part weighing 5,000 lb. and a stroke of 3½ ft. The piles were of Douglas fir, 80 to 100 ft. long, with the small end sawed off square, and were driven in 40- to 50-ft. depths of water without the use of a follower. The tips of the drop-hammer piles averaged 8.1 in. in diameter, and the depth driven into the ground ranged from 18.2 to 48.9 ft., averaging 30.7 ft. The tips of the steam-hammer piles averaged 9.1 in. in diameter, and the depth in the ground ranged from 24.8 to 38.9 ft., averaging 30.1 ft.

The average penetration in inches per blow for each 4 ft. of driving is given in the accompanying table.

Depth in Ft.	Penetration in In.	
	Steam	Drop
0-4	2.37	5.4
4-8	1.46	3.7
8-12	1.29	3.2
12-16	0.86	2.38
16-20	0.77	1.79
20-24	0.55	1.14
24-28	0.37	0.85
28-30	0.29	0.27

The safe bearing values of the drop-hammer piles computed by the *Engineering News* formula $F = \frac{2wh}{s+1}$ and of the steam-hammer piles computed by the formula $F = \frac{2wh}{s+0.3}$ correspond fairly closely when reduced to corresponding penetrations, as shown on the accompanying diagram. By referring the drop-hammer values to what may be termed standard driving, using $w = 3{,}000$ lb., $h = 20$ ft., and s proportional to the observed penetration in the ratio of $\frac{3{,}000 \times 20}{3{,}460 \times 12}$, the accordance between the two sets of bearing values is considerably closer. When the steam-hammer pile values are computed by the formula $\frac{2wh}{s+0.1}$ (the generally accepted steam-hammer formula), the results vary considerably from the above and are on the unsafe side. Apparently the use of this formula leads to the bearing values that are excessive.

Somewhat similar conditions were found in driving test piles for the Mare Island dry dock in 1906, where the average pile penetration per blow under the 29-ft. fall of a 2,600-lb. drop hammer was 0.86 in., which corresponded to a final penetration of 0.3 in. for the steam hammer with a 5,000-lb. moving part and a 41-in. stroke. The safe load by the usual drop-hammer formula referred



COMPARATIVE BEARING VALUES OF TEST PILES WITH STEAM AND DROP HAMMERS

to a 20-ft. fall of a 3,000-lb. hammer is 70,000 lb. By the steam-hammer formula $\frac{2\,wh}{s + 0.3}$ the safe load is 58,000 lb.; by steam-hammer formula $\frac{2\,wh}{s + 0.1}$ the safe load is 87,500 lb., which is apparently excessive.

At Pearl Harbor the average penetrations per blow by steam hammer and by drop hammer at the same attained depth were almost exactly proportional to the respective energies of the blow. At Mare Island this relationship, as applied to the two kinds of hammers, was not so close, as the ratio of the respective penetrations was 0.35 and the striking energies 0.23. The steam hammer, probably because of its greater weight, gave about 50% more penetration in proportion to the energy expended; but the penetration, 0.86 in. per blow of the 29-ft. fall of the drop hammer, was very closely in proportion to the penetration 0.34 in. per blow of the same hammer with an 11-ft. fall.

NOTES

Area in Acres—Twice the area in square yards, and the decimal point shifted four places to the left (or, as the author states it, twice product of dimensions in hundreds of yards), gives the approximate number of acres. The result is 3.2% small, and may be corrected by adding 1/30 of the result. Thus, with an area a mile long and ¾ mi. wide, the dimensions are 1,760 yd. and 1,320 yd., or in hundreds of yards 17.60 and 13.20. The product is 232.32, and twice this is 464.64. Adding a correction of 1/30, or 15.488, gives 480.13 acres. The true area is 480 acres. This rule, given in "Professional Memoirs" of November-December, 1915, is credited to G. E. Gibson, Assistant Engineer, Barge Canal Office, Albany, N. Y.

Top Forms in Arch Concreting—Top forms are considered necessary by H. B. Watters, resident engineer on the new Chesapeake & Ohio Northern Ry. work at Sciotoville, Ohio. It has long been usual in concrete bridgework to omit top forms near the crown of the arch where the slope is very small, but of late it has become rather common practice to omit these forms far down the arch rib slope and to attempt to screed the top surface to line. Concrete so placed is apt to pile up, causing an excess of arch-ring depth at the bottom of the section poured and a thinning of the ring near the crown. To avoid this dangerous condition Mr. Watters uses a top form for the haunch section of the arch ring, which extends from the springing line to a crown section placed in advance about 3 ft. on either side of the crown.

Tables for Estimating Stone or Concrete Trim have been compiled by W. E. Corbett, of Corbett & Clarke, Hackensack, N. J., and are presented herewith. The figures in the first horizontal and the first vertical line in both tables are the cross-sectional dimensions of the construction to be estimated. The weight or volume per lineal foot of a piece of 6x6-in.

TABLE 1. WEIGHT OF TRIM, POUNDS PER LINEAL FOOT

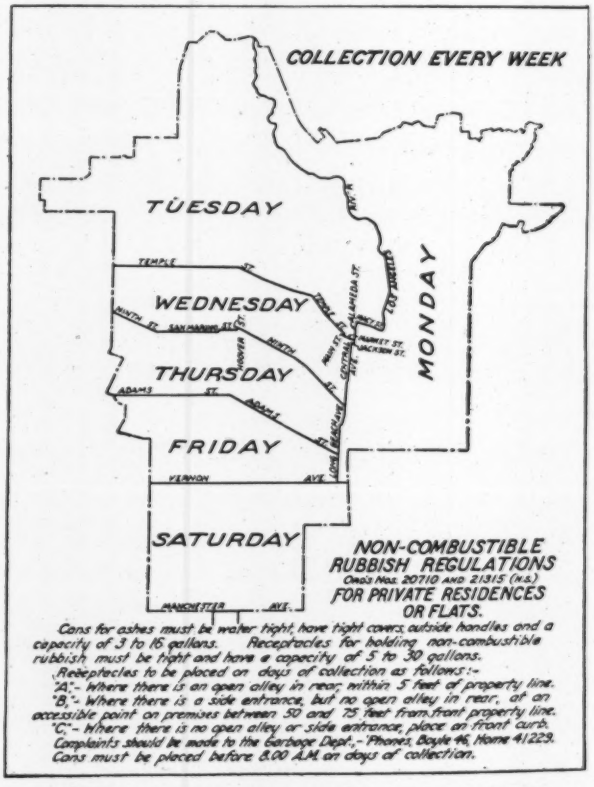
Sizes, In.	4 In.	5 In.	6 In.	7 In.	8 In.	9 In.	10 In.	11 In.	12 In.
4	17	21	26	29	33	38	41	45	50
5	21	26	32	36	42	47	53	57	63
6	26	32	38	44	50	57	63	69	75
7	29	36	44	51	59	66	74	81	87
8	33	42	50	59	66	74	84	92	99
9	38	47	57	66	74	84	93	104	113
10	41	53	63	74	84	93	104	114	125
11	45	57	69	81	92	104	114	126	138
12	50	63	75	87	99	113	125	138	150

TABLE 2. VOLUME OF TRIM, CUBIC FEET PER LINEAL FOOT

Sizes, In.	4 In.	5 In.	6 In.	7 In.	8 In.	9 In.	10 In.	11 In.	12 In.
4	0.11	0.14	0.17	0.19	0.22	0.25	0.27	0.30	0.33
5	0.14	0.17	0.21	0.24	0.28	0.31	0.35	0.38	0.42
6	0.17	0.21	0.25	0.29	0.33	0.38	0.42	0.46	0.50
7	0.19	0.24	0.29	0.34	0.39	0.44	0.49	0.54	0.58
8	0.22	0.28	0.33	0.39	0.44	0.49	0.56	0.61	0.66
9	0.25	0.31	0.38	0.44	9.49	0.56	0.62	0.69	0.75
10	0.27	0.35	0.42	0.49	0.56	0.62	0.69	0.76	0.83
11	0.30	0.38	0.46	0.54	0.61	0.69	0.76	0.84	0.92
12	0.33	0.42	0.50	0.58	0.66	0.75	0.83	0.92	1.00

trim, for example, can be determined by moving to the right on the horizontal line which starts with 6 in., until in the column headed by 6 in. the answer will be found—0.25 sq.ft., or 38 lb., as desired. The weight table is prepared on the basis of 1 cu.ft. weighing 150 lb.

Refuse-Collection Card with District Map—For the convenience of householders a district map is printed on the refuse-collection instruction cards issued in Los Angeles, Calif. The map shows what parts of the city are served each


COLLECTION EVERY WEEK
TUESDAY
WEDNESDAY
THURSDAY
FRIDAY
SATURDAY
MONDAY
NON-COMBUSTIBLE RUBBISH REGULATIONS
Ords Nos. 20710 AND 21315 (N.S.)
FOR PRIVATE RESIDENCES OR FLATS.
Cans for ashes must be water tight, have tight covers, outside handles and a capacity of 3 to 15 gallons. Receptacles for holding non-combustible rubbish must be tight and have a capacity of 5 to 30 gallons.
Receptacles to be placed on days of collection as follows:-
'A'- Where there is an open alley in rear, within 5 feet of property line.
'B'- Where there is a side entrance, but no open alley in rear, at an accessible point on premises between 50 and 75 feet from front property line.
'C'- Where there is no open alley or side entrance, place on front curb.
Complaints should be made to the Garbage Dept., Phones, Boyle 96, Home 41229.
Cans must be placed before 8.00 A.M. on days of collection.

REFUSE-COLLECTION MAP, LOS ANGELES, CALIF.
(The original was 5x6½ in.)

working day of the week under the prevailing weekly collection of noncombustible refuse. The card also gives excerpts from the ordinance governing receptacles for ashes and for rubbish.

Editorials

A Railway Freight Blockade Without Precedent

Nine years ago, in the winter of 1906-7, a congestion of railway freight traffic occurred which extended over a large part of the country and produced the most serious freight blockade ever recorded in American railway history. The chief cause of that blockade was the fact that in many parts of the country railway terminal facilities were inadequate to handle the extraordinary volume of traffic which was thrust upon them during a period of unprecedented business prosperity. Secondary causes were that many railways were short of motive power at that time and short also of freight rolling stock.

On many of the railway lines east of Chicago the blockade of 1906-7 is being repeated today, as a result, however, of causes entirely different from those which produced the blockade of nine years ago. There has been for the past twelve months a very heavy railway traffic from interior points to Atlantic Coast ports for export shipment. In order that the cars carrying this export freight may be promptly unloaded when they reach the port for which they are destined, ships must be ready there to receive it.

Ever since the outbreak of the war the demand for shipping facilities has been greater than the available ocean vessels could supply. The National Foreign Trade Council recently estimated that the total tonnage of merchant vessels rendered unavailable for private commerce since the war began is between 20 and 25% of the total tonnage at the outbreak of the war. To some extent this reduction in the available shipping has been offset by a reduction in the amount of international trade. It must be remembered that great as is the amount of traffic now passing from America to Europe, there are other ocean shipping routes on which trade is at present far below the normal.

It is obvious, however, that there are not enough ships to supply the demand, and the condition is aggravated by the delays in unloading vessels in ports on the other side of the ocean.

In no industry are prices more promptly responsive to the law of supply and demand than in the ocean-freight business, and rates charged for ocean carriage are double and triple, in some cases even five to seven times what they were before the outbreak of the war. Even at these rates vessels cannot be obtained fast enough to supply the demand at the busiest Atlantic seaports, and for some time cars have had to wait weeks and even months before room could be found on board ship for their contents.

Fortunately the railway managers learned a lesson from the experience of nine years ago, and they put in operation, none too soon, an embargo against further shipments of freight to the points of congestion. Experience has abundantly demonstrated that if a railway allows its yards to become filled with cars beyond the point where free movement can take place, traffic will soon come to a standstill. The railway must keep tracks enough open for the movement of its passenger traffic and for the handling of perishable freight and livestock, and for the delivery to the communities along its line of such essentials of daily living as milk, meat, flour, fuel, etc. If it allows other classes of freight to accumulate in its yards, it will soon create just such a deadlock as afflicted a number of prominent railway centers in 1907.

It is an interesting question whether the railway facilities of the present day would be sufficient for the present volume of traffic if sufficient ships were available at the seaports promptly to take care of the export business. There has been, it is true, an enormous extension of terminal facilities since 1907. Had there not been, the natural growth of traffic since that time could not possibly have been taken care of. During the last two or three years, however, the prevailing sentiment in railway circles has been that fully as much money had been invested in terminals as business would warrant, and the long periods since 1907 when there has been an enormous surplus of unused cars have had a tendency to keep down investments in rolling stock.

It is of interest in this connection to note that in an official statement addressed to the Interstate Commerce Commission on Dec. 28, the New York, New Haven & Hartford R.R. declares that it is doing the largest volume of business in its history and has reached a point where its present physical facilities make it unable to handle a larger volume of traffic. It frankly admits its responsibility to the public for increasing these facilities, and announces that improvements to its lines costing $5,000,000 will be made as rapidly as men, material and weather will permit; that it has 33 heavy freight locomotives to be delivered in January and February and 500 coal cars in March, and that it is ready to spend in addition very large sums for more tracks, bridges, terminals, shipping facilities and motive power if money can be obtained.

Railway Boat Lines

As many of our readers will recall, a clause in the Panama Canal act forbids a railway company to operate vessel lines directly or indirectly, except by special permission of the Interstate Commerce Commission. The reason for this provision was the idea that if the railways were allowed to run their own steamship lines through the Panama Canal they would first cut rates to a point which would ruin competitors and then raise them to such a figure that the canal route would not seriously compete with the rail lines across the continent. It is claimed that such tactics have been actually used by coastwise steamer lines operated in connection with railways to drive off competition.

Whatever of truth there may have been in this argument in the past, experience of the last few years appears to demonstrate that independent transportation companies which engage in coastwise traffic do not fur-

nish facilities and service equal to that given by the railway-owned steamer lines. Shippers and travelers prefer to patronize a boat line run by a responsible railway company in preference to a line of whose responsibility they know little or nothing.

For many years the service on Long Island Sound furnished by the boat line operated by the New Haven R.R. was famous for its excellence. In recent years, as might naturally be expected, the railway company has given little attention to its boat service. It has not, however, ignored the possibilities of development in this direction, for President Elliott says, in the statement above referred to:

> The company has tried to relieve its rail lines so far as it could by diverting business to its boat lines, but here again it has not been able to add to its facilities because of the uncertainty surrounding the future of the boat lines. The company considered very seriously placing an order nearly a year ago for two additional freight steamers to run on Long Island Sound, but it did not feel that it had the right to borrow the large sum of money necessary, considering the fact that within a comparatively short time the company might be ordered, under the Panama Canal Act, to dispose of all of its water lines. The relief, therefore, given by the boats is not as great as the company would like.

There is a reason why capitalists naturally hesitate before investing a large amount of money in coastwise steamship lines to compete with railways. In order that any shipping line shall do a profitable business it must have the coöperation of connecting railway lines. It is probably within the power of state and interstate commissions to compel railways to grant certain facilities, such as through routes and through rates, to a coastwise steamship line; but coöperation produced by compulsion is not apt to be effective and valuable coöpertion. The law can compel certain acts to be performed, but it cannot possibly deal with the thousand and one details which are necessary if a railway is effectively to aid a steamer line to get business instead of encouraging travelers and shippers to use the rail line.

On the other hand, if a railway company were permitted to operate steamship lines, there is certainly sufficient authority in government supervising commissions to see that proper facilities are furnished and at reasonable rates.

An Engineer Reviews His Work

Few men who give their names to a marked engineering advance are able to lead that advance so far and so ably as has Dr. Karl Imhoff. Eight years ago, realizing some of the drawbacks to the septic-tank process of sewage clarification and sludge reduction, and with keen appreciation of the improvements in this process suggested by the Massachusetts State Board of Health and partly worked out by Dr. Owen Travis, Dr. Imhoff designed a new type of two-story tank. As engineer of the Emscher Sewerage District, in Germany, Dr. Imhoff had immediate opportunity to install a number of these tanks. Promptly and enthusiastically introduced into the United States by Dr. Rudolph Hering, the Imhoff, or Emscher, tank has since been widely used here, although not so extensively as in Germany. Today a quarter-thousand Imhoff tanks are in use or under construction in the two countries.

Just how many of these tanks have been designed by or under the more or less close supervision of Dr. Imhoff cannot be stated. Unquestionably he has had some part in a large percentage of them, and engineers who have been inspired by personal touch with him have designed many of the others. Broadly speaking, the type has been followed more closely than is usual in the absence of rigid proprietary control. The large number of two-story tanks which he has himself designed, his closeness of touch with the work of others and the widespread and growing use of the Imhoff tank combine to give great interest and value to his review of "Eight Years of Imhoff Tank Design and Operation," begun elsewhere in this issue. The review is worth reading by not only those interested in sewage disposal, but also by engineers of sufficient breadth to desire to follow the outworking of a problem in engineering design and operation.

Engineers and Bridge Art

For an engineer to pretend to have any authoritative knowledge of art in construction is of course an impertinence. That privilege is the prized possession of the allied profession of architecture. The appearance of a bridge, however, is so important for its success that an engineering journal may presume to offer some humble comments upon it.

On the first page of this issue is a view of the Q St. bridge in Washington, D. C., the latest of several arch bridges which span Rock Creek in that city. In such a black and white print the colorful tones of the bridge are lost and the prodigality of decorative detail can hardly be appreciated. This is unfortunate, for it is in these two features that the bridge differs most radically from other bridges which have been built by engineers without the aid of architects. In the Q St. bridge the decoration of the bridge was intrusted to an eminent architect.

There is a widespread impression that engineers are purely utilitarian, that they are too much concerned with strength and economy to consider appearance and that, in the minds of the profession, strength and ugliness are twin sisters. There may have been some ground for this impression years ago, particularly in steel-bridge design, but in the design of the masonry arch most engineers have learned that simplicity generally means beauty and that that arch which best satisfies the structural requirements will as a rule best satisfy the eye. Most engineers therefore have hesitated to attempt to improve on the simple lines and tones of the concrete arch by either decoration or coloring.

In the Q St. bridge, however, an architect has attempted what no mere engineer would have dared. There are an elaboration and a profusion of ornament and tone which may possibly delight the artist or the architect, but which are not altogether pleasing to the engineering eye, accustomed to simplicity of detail. There is of course ample precedent in Europe for profuse bridge decoration. Practically every one of the many bridges crossing the Seine in Paris is bedecked with statues and basreliefs. Artists and art critics frequently bewail the absence of similar decorative bridges in American cities, which might lead one to believe that the American engineer is too commonplace in his ideas and that a conception of the beautiful is as far from his ability as the production of it.

It will hardly be disputed, however, even by the artists and art critics, that if the choice in bridge architecture

between simple plain outlines and a profusion of over-decoration, the simple style is far preferable. An engineer viewing the Q St. bridge will generally have no regrets that he has not ventured to hang decorations on his bridges. To his possibly untrained eye, the coloring is too emphatic, the decorations too obtrusive and the arched sidewalk brackets too fantastic. When compared with the simple lines and tones of the nearby Connecticut Ave. bridge—also artistically supervised by an architect—the newer bridge appears almost tawdry.

Individually each buffalo on the Q St. bridge seems to be a pretty good animal as bronze animals go, but collectively there are too many of them and they are too large for comfort. The impression is unavoidable that a whole herd of this nearly extinct species has selected an entirely unsuitable feeding ground and that a very small bridge has been inadvertently left underneath some very large bison. Incidentally, those who have wondered why animals—and wild animals at that—are artistic necessities of bridge decoration may be interested in the bridge pictured on page 40 of this issue. The Chinese school of art—wild dogs, sculptured lions and all—has evidently bridged both time and space and is influencing our modern architects.

Developments in Surveying

It is not very frequently that the editor can direct attention to new methods of surveying, but on another page the reader will find an article describing some truly radical improvements in the field work of topographic surveying. While this article deals specifically with reconnoissance surveys for logging operations, it will be readily seen that the method is much more accurate than work hitherto done for this purpose, and there is no reason why the same methods cannot be applied to railway reconnoissance or to any survey requiring topographic details of ordinary accuracy.

The new method involves the use of a special Abney clinometer and a specially graduated tape. With this clinometer the vertical rise per chain (66 ft.) is read directly, and used in conjunction with the special tape, it permits stakes or markers to be set at unit intervals of horizontal measurement, quickly and without computations or the use of tables, irrespective of the slope to which the tape is applied.

Here are all the elements necessary to locate contours except the horizontal control. In thickly wooded forest land this control is obtained by a series of parallel lines run out by a compass, but when desirable, any other system of control may be adopted. Experience shows that elevations may be made to check within 1 ft. per mi. (10 ft. per mi. is the average for rough work), so that a more accurate system of control than compass lines is justified in an open country.

Incidentally the attention of the reader is called to the articles in *Engineering News,* June 10 and Dec. 16, 1915, which give in considerable detail the methods used and the accuracy obtainable by the use of an aneroid barometer for topographic reconnoissance. The former article describes the use of aneroids for forest surveys and the second article describes the use of a single aneroid for a very rough reconnoissance. These two articles and the one in this issue comprise a very complete and uptodate treatment of this important branch of surveying.

Model-City Planning Boards

If cities are to be planned instead of happening, someone must be charged with the responsibility of the task. A half-dozen years ago Hartford, Conn., under the leadership of its city engineer, created the first city-planning commission in the United States. Today there are perhaps a hundred such commissions, East and West, North and South. Naturally in the earlier unchartered stages of city planning control and under our multifarious forms of city government, the composition, powers and duties of American city-planning commissions and the relations of such commissions to other branches of the city government were most varied. It will take some time to find out just what is the best method of control—or perhaps the three or four best methods to suit varying conditions.

An attempt to outline a scheme for city-planning boards under the commission-manager form of city charter has just been made by a committee of the National Municipal League. Without repeating the details, printed elsewhere in this issue, attention may be called to these important features of the scheme of particular interest to engineers: (1) Everything affecting the city plan must be passed upon by the board. (2) Every year the board must submit a city-planning program for at least three years to come. (3) The city engineer is recognized as the chief city planner in ordinary, so to speak, but the board must employ as secretary a person skilled in city planning and may engage the services of consulting experts in city planning. All these features are commendable.

The engineering profession throughout the country will heartily approve the selection by Governor Whitman of Henry W. Hodge as a member of the New York City Public Service Commission. For several years engineers have discussed the need of appointing men of engineering experience and demonstrated executive ability as members of state public-service commissions, and organized efforts to secure such appointments have been made by the engineering societies. The selection of Mr. Hodge for this position by Governor Whitman is in line with these recommendations of the profession. More than all else, the public is to be congratulated that an office of such importance is to be filled by a man in every way competent to discharge its duties.

The letter from General Kingman, on the next page, reports a definite step forward in the Corps of Engineers of the United States Army, a step which must be pleasing to the civilian engineers of the country and one that has been urged in public and in private for many years. It is true that some years ago the way was made clear to permit the commissioning of capable young engineers from civilian life, but the rarity of such appointments was such as to foster the suspicion that the more-or-less intentional choking off of civilian recruiting was to extend from the civil-engineering corps of the navy to the similar corps in the army. The success of the latest army examinations and the methods used to get candidates for those examinations show that the present administration of the Engineer Corps at least is determined to get good men where they can be found and not to depend alone on West Point, which, at best, is not an engineering school.

Letters to the Editor

Sharp Thread Caused Failure of Suspender Rods

Sir—I have been interested in the description of the failure of the Division St. Bridge at Spokane, Wash., as published in your issue of Dec. 30, 1915, and think that Hugh L. Cooper has probably reached the correct conclusion—that the cause of failure was the treatment of the eye-bars after the accident of 1894.

I would call attention, however, to the comparison in your editorial to the breakage of suspender rods in the Brooklyn Bridge. The breakage of these rods, as I remember, was not due to any heat-treatment after they were manufactured, but rather to the improper shape of the screw thread as originally cut. These suspenders were among the first to be made of steel in bridge construction and before it was generally recognized that a sharp cut in homogeneous steel is equivalent to an incipient crack that will creep through the steel when the metal is repeatedly strained. The threads were cut with sharp angles (no fillet at bottom), and the fracture showed by concentric rings of rust that the suspenders had broken gradually until so small a section remained intact that it could not then stand the load imposed.

When one suspender broke, it increased the load on adjacent suspenders, and several others broke in succession. HENRY B. SEAMAN.

Woolworth Building, New York, Dec. 31, 1915.

Six Civilians Commissioned in Army Engineer Corps

Sir—I take pleasure in sending you the following information:

As a result of the examination for appointment as probational second lieutenant in the Corps of Engineers, United States Army, held on Oct. 25-30, 1915, the nominations of the six successful candidates are about to be sent to the Senate. These are Messrs. Gilbert D. Fish, New York City; George Mayo, Washington, D. C.; Paul T. Bock, New York City; Charles O. Boynton, San Francisco, Calif.; Arthur P. von Deesten, Brooklyn, N. Y.; and Edward N. Whitney, Madison, Wis. Mr. Boynton is a junior engineer who has been several years in the Government service, under the Engineer Department.

The number of candidates who actually took and completed the examination was 17. In addition three candidates withdrew during the progress of the examination, while two were found physically ineligible as a result of the physical examination which preceded the mental examination.

The nomination of the six successful candidates marks the completion of a vigorous effort on the part of the Chief of Engineers of the army to secure thoroughly well-qualified young engineers for appointment to the Corps of Engineers under the provisions of the river and harbor act approved Feb. 27, 1911.

The number of candidates presenting themselves for this examination in previous years was disappointingly small, and very few of the candidates who presented themselves had the necessary qualifications. In the period from January, 1912, to February, 1915, four examinations were held. The total number of persons who took the examinations was 11; of these three were commissioned as probational lieutenants.

In order to secure a larger number of candidates, the Chief of Engineers on Apr. 1, 1915, addressed letters to the presidents of all the approved technical schools, about 50 in number, requesting that they draw the attention of suitable graduates of their institutions to the opportunity for appointment as lieutenants in the Corps of Engineers. The Chief of Engineers also issued instructions to all of the District Engineer officers in the United States to facilitate in every way the candidacy of junior engineers in the employ of the Engineer Department and of any other candidates who might make inquiry regarding the examinations.

Arrangements were made with the Civil Service Commission by which a special examination was held to permit candidates to establish eligibility for junior engineer as required by law.

As a result 25 persons were authorized to take the examination beginning Oct. 25; of these, two withdrew before the examination and one failed to appear, leaving 22 who presented themselves for the examination, as already indicated.

It is anticipated that the next examination for appointment of the Corps of Engineers will be held at a convenient time after the termination of this college year.

DAN C. KINGMAN,
Chief of Engineers, U. S. Army.

Washington, D. C., Dec. 31, 1915.

Dishonest Hydraulic-Turbine Specifications and Bids

Sir—Bidders on turbines for hydraulic-power development are often asked to submit with their bids the "estimated approximate weights of the turbine units." Where, as is commonly the case in present practice, the equipment is specially designed, it is frequently impossible for the turbine manufacturer to estimate with precision the weights of the turbines at the time the bids are made. There is a temptation for the turbine manufacturer to overestimate the weights, for the reason that the purchaser is prone to think that the greater the weight of the machinery, the more he is getting for his money.

Every intelligent engineer will admit that this is a fallacy with reference to the purchase of almost any kind of high-grade machinery. It is certainly a fallacy in the case of hydraulic turbines. A certain amount of weight is absolutely necessary; but beyond this, if the design is correct, additional weight in the turbine runner, casing or gate mechanism is decidedly a disadvantage. Unneces-

-ary metal in the runner increases the load on the bearings and is quite certain to reduce the efficiency. Unnecessary weight in the gate mechanism increases the cost of governors and also increases the difficulty of close speed regulation. Unnecessary weight in the turbine casing or in any of the other parts increases the size of crane necessary to handle the weight; increases the cost of freight and installation, and in many cases increases the power-station building cost throughout.

The excessive weights can be dishonestly employed to throw a turbine contract to favored builders. Now it must be admitted that engineers from time immemorial have been accustomed to resort to devious, and sometimes highly questionable, methods to eliminate undesirable bidders on contract work, though such practices have been carried out from good motives; identically similar methods may be utilized unfairly to throw work to favored bidders. In the case of hydraulic turbines, acceptance of the bid is often made contingent on several factors: First, efficiency, which may be estimated in various ways; second, net price bid. This as ordinarily interpreted would be supposed to mean the gross sum bid for the work. But suppose that one bidder overestimates the weight of the equipment, say 40%, not really intending to put this additional weight into the work, and that the other bidders estimate the approximate weights within say 10% of the true weights which they will furnish. The result is likely to be that the bid which is based on excessive weights will show an apparent cost per pound lower than that which any bidder whose weights are honestly given can afford to bid. The work may then be awarded to the highest bidder on the ground that the net price per pound is lowest, even though the efficiency guarantee is not above that of other bidders whose gross price is lower.

Examples of just this practice have come to the writer's attention, and it has seemed worth while to suggest a remedy. While a prospective bidder may not be able to tell in advance the exact weight within say 10% or 15%, he can at least tell what the minimum weight necessary will be. Accordingly it would seem that hydraulic-turbine specifications should require that bidders guarantee to furnish equipment having a certain minimum weight; and in order to prevent the furnishing of equipment of excessive weight, they should also be called upon to guarantee that the weight will not exceed the guaranteed minimum by more than a certain percentage, 10%, 15% or 20%, according to conditions.

This plan will at least furnish a uniform scale of comparison between bids, while under the present practice several bidders equally honest in their intentions may add various percentages to their estimated weights in order to be on the safe side. If this practice was generally followed, then the omission of such a weight guarantee clause from the specifications would in effect serve notice upon those proposing to bid on the work that there is likely to be an Ethiopian in the hedgerow.

CONSULTING ENGINEER.

A Coal-Thawing Shed will be built in connection with the coal-shipping pier of the Baltimore & Ohio R.R. at Curtis Bay, Baltimore, Md. Here cars of coal will be placed, in cold weather, before being run out to the car-dumping machines on the pier. The pier will have a capacity for shipping 10,000,000 tons per year and will cost about $1,500,000. It is to be completed before the end of this year.

Centrifugal Sewage Pump Saves Force Main

BY GEORGE G. EARL*

The new electrically driven centrifugal pump at New Orelans Sewerage Station A (designed by A. B. Wood, Mechanical Engineer of the Sewerage and Water Board, and purchased under the same contract as the large screw pumps for drainage) has recently been found to develop a capacity of 55 to 133 cu.ft. per sec. against heads of 43 to 72 ft. at efficiencies of 69 to 84%. This pumping unit has postponed for some years the construction of an additional force main. It is driven by a two-speed induction motor, takes water from the main sewer and discharges it through a 48-in. force main, 7,000 ft. long, into the Mississippi River. At low speed it makes 248 r.p.m. and discharges, depending upon the elevation of water in the river and in the sewer, from 55 to 85 cu.ft. per sec., showing an efficiency ranging from 69 to 83% on lifts (including friction in the discharge main) ranging from 43 to 35 ft. At high speed it makes 364 r.p.m. and discharges, depending upon the elevation of water in the river and in the sewer, from 106 to 133 cu.ft. per sec., showing an efficiency ranging from 80 to 84% on lifts (including friction in the discharge main) ranging from 87 to 72 ft.

These efficiencies are based in this case on heads shown by the pressure difference on the two sides of the pump and are slightly higher than was expected with a pump designed for two different speeds and two different conditions of lift, widely separated.

HOW EQUIPMENT IS SAVED

There were originally installed at this station two engine-driven centrifugal pumps which were each capable of discharging 58 cu.ft. per sec. through the 48-in. force main with 51% efficiency. It was intended in the original design of the station to add later another steam-driven pump of larger capacity and to lay an additional force main to the river, since the original engines could not work together with the existing force main, thus bringing the capacity of the station to something over 128 cu.ft. per sec. with one engine and pump in reserve. This would have cost, for force main and additional engine and pump of 70 cu.ft. per sec. capacity, at least $135,000.

By substituting the new pump and motor at a cost of $29,000 we have added a unit whose capacity can be either 75 or 120 cu.ft. per sec. and which is of much higher efficiency than the existing steam pumps, and postponed for several years the construction of an additional force main which would have cost about $100,000 for a 48-in. main. The crowding of 106 to 133 cu.ft. per sec. through the existing force main creates a materially higher lift than would have been practicable for an engine-driven single-stage centrifugal pump, but the efficiency of the electrically driven unit operated from a central power station is so high that this additional lift requires little if any more fuel than would have been required if the additional outlet line and a steam-driven pump had been adopted. The need for the high-speed drive exists mainly during and immediately after storms, when the sewers are overcharged with ground water. The low speed is sufficient for dry-weather flow.

*General Superintendent, Sewage and Water Board, New Orleans, La.

Bridge Art at Its Origin

The arch bridge pictured on this page is one of many years' standing, located in the Imperial Summer Palace near Peking, China. It is shown here as an example of old Chinese bridge architecture, and incidentally to indicate a possible source of the wild-animal style of bridge decoration so well exemplified in the first article in this issue. The Chinese wild dogs compare favorably with the Washingtonian buffalo.

Public Concern in Water Power*

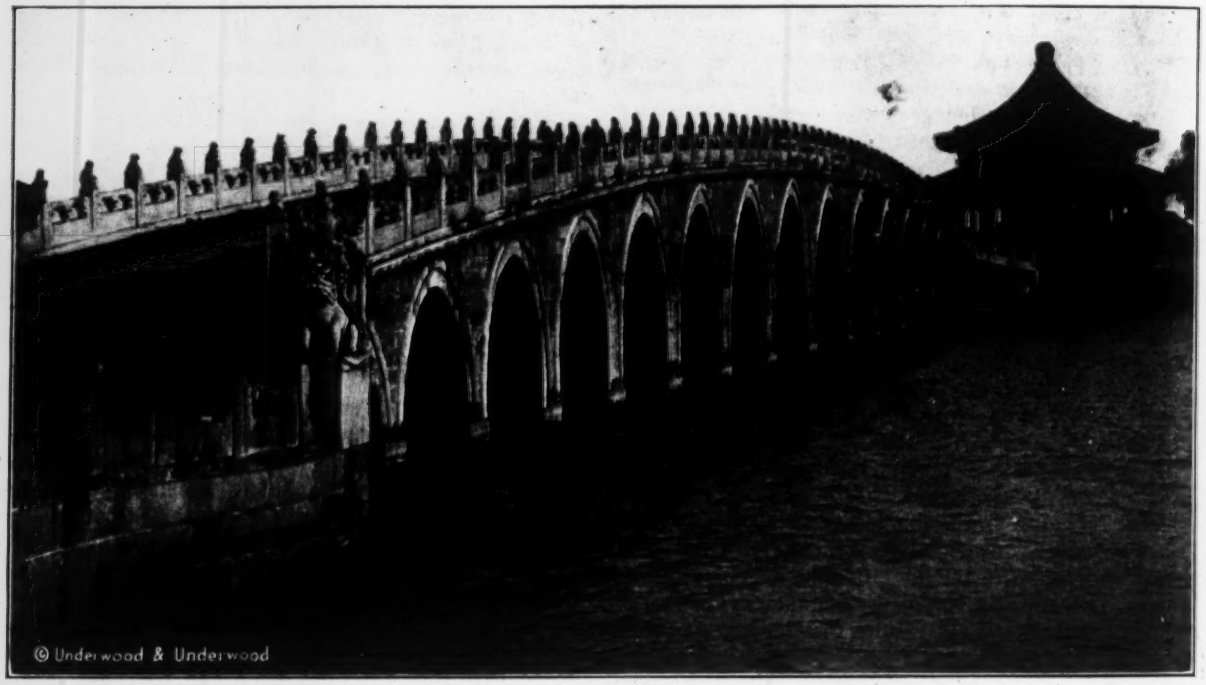
By George Otis Smith†

The people's interest in water power is served only through use. Whenever the energy of flowing water can be put to work at a cost justified by results, nonuse of a water power is an economic waste. But better not even a partial development now, if thereby the full utilization that may be needed in the near future is blocked. From the standpoint of the public what is needed in water-power utilization is efficient development that will meet market demands, and effective regulation that will secure to all parties interested a participation in the benefits of that development.

It is this logical desire for the best use, in which the largest possible number may share, that makes the utilization of water power a live issue. Unwarranted fear of impending monopoly may have led some to urge a halt in all development, while blind disregard of a changing order of things has caused others to denounce any restriction.

The engineering fact that stands out in the utilization of water power is the comparatively high efficiency of large systems. The combination of several water powers differing in character in order to serve many markets having diversified service requirements makes possible an economic adjustment of supply to demand. Nature's irregularities are thus equalized and man's varied requirements are similarly balanced. The Western states, in which these large hydro-electric systems have been mainly developed, show the largest per capita consumption of power and the lowest average rates.

In Montana one mining company by substituting hydro-electric power for steam effected an annual saving of nearly two million dollars. This means a saving of about 50c. on a ton of ore, thus permitting the mining with an equal profit of ore that much lower in grade than could be mined when steam-generated power was used. This instance illustrates the wide public interest that may be served through the efficient operation of large units. There exists a certain community of interest between the power corporation and the mining company. A similar community of interest needs only to be created between the public-utility company and the people it serves.



MARBLE BRIDGE IN IMPERIAL SUMMER PALACE AT PEKING, CHINA

The oft-debated question whether the relation of the Federal Government to water-power sites is that of proprietor or that of sovereign is only incidental to the real issue of the people's larger interest as the consumers of power. Herein lies the purpose of certain legislative proposals that have been misunderstood by some and perhaps misrepresented by others. The method of control may be based upon public ownership of land, police powers or preservation of navigation; the primary results sought will be public protection.

Too often in the past the wealth won by the engineer from the mine, the forest or the stream has been monopolized by the capitalist. Experience is teaching the American people to regard industrial development as a public

*From an address at the second Pan-American Congress, Dec. 27, 1915, to Jan. 8, 1916.

†Director, United States Geological Survey, Washington, D. C.

matter in which others than owners of stocks and bonds are concerned.

Too often the public interest in water-power resources has been discussed almost wholly in terms of compensation, tolls and charges for the publicly owned resources utilized. The revenue idea connected with this conservation issue has been allowed to overshadow the thought of control in the public interest. A notable exception to this tendency is contained in the Pend d'Oreille permit issued in 1913 by Secretary Lane, which provides that after the ten years allowed for construction of plant and development of market the rate of compensation to the Government shall vary inversely as the square of the proportional development of the site and directly as the square of the average price charged for electric energy to both customers and consumers. Here the objects plainly sought are prompt and complete development and low rates to the public rather than revenue to the Government.

When applied to water-power resources, as to other public utilities, regulation by public agencies needs to begin in the initial stages of development. The terms of the contract between the public consumer of electric energy and the private builder of the hydro-electric plant can best be considered before a stone is laid.

In any discussion of the future conduct of the water-power business one of the essential premises is the condition of proper return. This idea is at the foundation of public regulation of private business; but even beneath this is the query, A fair return upon what? The present trend in the determination of the proper basis for rate regulation appears to be away from valuations derived either from cost reproduction or from capitalization and toward actual legitimate investment.

The practical ideal in public regulation is to allow such returns to the private owner and operator as may encourage engineering and executive efficiency. To fix a maximum rate of profit based simply on capital investment might effectually veto improvement in plant or operation beyond the dead-line thus established; therefore the prize of a larger return must be held out to those who make the largest and best use of the natural resource. In the absence of the competitive element the returns to be derived by the private holder of a natural monopoly should be restricted so far as they depend primarily upon his use of the resource which society owns or to which society gives value; but a quite different method of regulation may well be applied to the profits that come from his continued investment of gray matter. Such ability and industry need not be capitalized, yet betterment of service should produce benefits in which both investor and consumer may share.

Adequate incentive to greater efficiency combined with full recognition of the people's interest may be provided by any of several methods—the sliding-scale plan adopted for the Boston gas companies makes larger dividends to stockholders contingent on lower prices to consumers; the Chicago street-railway ordinance provides for the division of profits above the fixed rate between the company and the public; a profit-sharing plan worked out by the manager of Wisconsin power company divides excess net profits equally between the company and the consumers. In each of these plans the principle is the same—the investing public and the consuming public are partners in the enterprise, and the duty of the Government is to exercise its protective power in the interest of the private owner of a public utility no less and no more than in the interest of the public user of that utility.

City-Manager Plan Adopted by 75 Municipalities

Large and small, 73 cities and towns in the United States and 2 in Canada have adopted the city-manager idea of municipal government. By this time the number may be larger, since the list published herewith was compiled some weeks ago and accessions are made at frequent intervals.

Two facts not disclosed by the table are that most of the places in the list combine the city manager and commission plans, and many of the cities and towns employ engineers as city managers.

MUNICIPALITIES UNDER CITY-MANAGER GOVERNMENT IN THE UNITED STATES AND CANADA

(As Reported to Ossian E. Carr, Secretary City Manager's Association, Up to November, 1915)

City	Population 1910	Date in Effect	Manager	Annual Salary
Norwood, Mass.	8,014	1914	C. A. Bingham	$3,000
Niagara Falls, N. Y.	30,445	Jan. 1916	O. E. Carr*	5,000
Newburgh, N. Y.	27,805	Jan. 1916	F. C. Alber*	6,000
Watertown, N. Y.	26,730	Jan. 1918		
Grove City, Penn.	3,674	1914	J. S. Ekey	2,000
Titusville, Penn.	8,533	Dec. 1913	H. A. Holstein	
Charlottesville, Va.	6,765	1913	A. V. Conway	
Fredericksburg, Va.	5,874		R. S. Royer	
Luray, Va.	1,218		Walter Campbell	
Portsmouth, Va.	33,190	Sept. 1916		
Staunton, Va.	10,604	1908	S. D. Holsinger	2,500
Wheeling, W. Va.	41,641	Jan. 1917		
Durham, N. C.	18,241			
Elizabeth City, N. C.	8,413	Apr. 1915		
Hickory, N. C.	3,716	May 1913	S. C. Cornwell	2,000
Morgantown, N. C.	2,712	May 1913	R. W. Pipkin	1,200
Thomasville, N. C.	3,877	1915	F. D. Jones	
Beaufort, S. C.	2,486	1915	H. C. Otis	
Rock Hill, S. C.	7,216	1915	J. G. Barnwell	2,400
Sumter, S. C.	8,109	Jan. 1912	Vacant	3,300
Lakeland, Fla.	3,719	1914		2,100
Largo, Fla.	291	1913	J. T. Sauls	600
St. Augustine, Fla.	5,494	1915	W. L. Miller	3,600
Johnson City, Tenn.	8,502	Jan. 1915	P. F. McDonald	
Ashtabula, Ohio	18,266	Jan. 1916		
Dayton, Ohio	116,577	Jan. 1914	Henry M. Waite	12,500
Sandusky, Ohio	19,989	Jan. 1916		
Springfield, Ohio	46,921	Jan. 1913	C. E. Ashburner	6,000
Westerville, Ohio	1,903	1915		
Alpena, Mich.	12,706	Apr. 1916		
Benton Harbor, Mich.	9,185	1916		
Big Rapids, Mich.	4,518	1914	W. J. Fairbairn	1,500
Cadillac, Mich.	8,375	Mar. 1914	O. E. Carr†	3,000
Grand Haven, Mich.	5,856	1914	E. M. Beck	2,500
Jackson, Mich.	31,433	Jan. 1915	G. C. Cummin	5,000
Manistee, Mich.	12,381	Apr. 1913	C. E. Ruger	2,000
Glencoe, Ill.	1,899	1914	H. H. Sherer	2,400
River Forest, Ill.	2,456	1914	Voted back to old form	
Winnetka, Ill.	3,168	Jan. 1915	R. L. Fitzgerald	2,400
Horicon, Wis.	1,881		E. C. Rehfield	1,000
Bloomfield, Iowa	2,028	1912	R. C. Bristow	680
Chariton, Iowa	3,794			
Clarinda, Iowa	3,832	1913	A. T. Wilson	1,700
Iowa Falls, Iowa	2,797	1914	E. L. Marriage	1,800
Webster City, Iowa	5,208	1915		
Morris, Minn.	1,685	1913	S. A. Siverts	1,600
Clark, S. D.	1,220	1912	J. E. Smith	960
Abilene, Kan.	4,118	1913	Kenyon Riddle	1,200
Mulberry, Kan.	997	1914	John Marion	
Amarillo, Tex.	9,957	Jan. 1914	M. H. Hardin	2,400
Denton, Tex.	4,732	1914	W. L. Foreman	
San Angelo, Tex.	10,321	Apr. 1916		
Sherman, Tex.	12,412	1915	K. M. Mitchell	
Taylor, Tex.	5,314	1914	Peter Schramm	
Terrell, Tex.	7,050	1913		
Tyler, Tex.	10,400	Apr. 1915		
Montrose, Colo.	3,252	1914	E. McDaniel	1,800
Collinsville, Okla.	1,324	1913	Claude Thorpe	2,000
Norman, Okla.	3,724			
La Grande, Ore.	4,843	Jan. 1914	F. J. Lafky	2,400
Snohomish, Wash.	3,244	1915	E. Colburn	
Roswell, N. M.	6,172	1914	W. M. Atkinson	1,800
Alhambra, Calif.	5,021	July 1915		
Bakersfield, Calif.	12,727	May 1915	Wallace M. Morgan	3,000
Glendale, Calif.	2,746		Thomas W. Watson	
Holtville, Calif.	729		D. A. Starbuck	
Inglewood, Calif.	1,536	1914	Abandoned	
San Diego, Calif.	39,578	May 1915	F. M. Lockwood	6,000
San Jose, Calif.	28,946	July 1916		
San Rafael, Calif.	5,934		W. A. Boland	
Santa Barbara, Calif.	11,659	Jan. 1917		
Phoenix, Aris.	11,134	Jan. 1913	R. A. Craig	5,000
Tempe, Aris.	1,473	1915		
Tucson, Aris.	13,193	1915	C. K. Clark	
Maisonneuve, P. Q.				
Port Arthur, Ont.				

* Appointed since list was compiled.
† Resigned to go to Niagara Falls since list was compiled.

News of the Engineering World

Rogers Pass Holed Through

The five-mile long Rogers Pass tunnel of the Canadian Pacific Ry.—the longest tunnel on this continent—was holed through on Dec. 19. The alignment, grade and distance were perfect. The Rogers Pass tunnel was notable from three standpoints—its length, the novel method of tunneling adopted and the rapid rate of advance.

In order to expedite the work, two pioneer tunnels were driven parallel to the main tunnel, but about 50 ft. distant from it. From these drifts, crosscuts were driven to the centerline of the main tunnel, and from these the main headings were driven.

The object of this method was to make the most rapid advance possible, and this has been accomplished. Some of the monthly advances made, in the first half of 1915, are: East pioneer heading, 776 ft.; east main heading, 538 ft.; east enlargement for double track, 726 ft.; west pioneer heading, 932 ft.; west main heading, 752 ft.; west enlargement, 812 ft. On July 25 the two pioneer headings were stopped when they were only 5,202 ft. apart. From these points the main headings were driven forward, meeting on Dec. 19, the average advance per month for each main heading having been about 520 ft.

The contractors for the tunnel are Foley Bros., Welch & Stuart, of Winnipeg; A. C. Dennis is their superintendent. The work is under the direction of John G. Sullivan, Chief Engineer, Canadian Pacific Ry., and W. A. James, Engineer of Construction.

Coffer Blown Up To Complete Troy Lock and Dam

On Dec. 31, 1915, the remaining section of the earth coffer-dam in front of the old dam across the Hudson River at Troy, N. Y., was blown up as a step in the last stages of the new lock and dam construction at that point. The view herewith shows the coffer at the instant of the explosion. The coffer-dam was built upstream of the old dam so as to permit the removal of that structure.

The new Troy dam is a concrete structure about 1,400 ft. north of the old dam and about 1,000 ft. long. It holds back a head sufficient to provide a minimum of 14 ft. of water over the sills of the new lock, which is 45 ft. wide and 492.5 ft. long. The dam and lock are being built by the Federal Government and are designed to supplement the New York State Barge Canal, which empties into the Hudson just above Troy. The entire project comprises the construction of the new lock and dam, the removal of the old lock and dam and the dredging of a 12-ft. channel for about 30 mi. below the entrance of the Barge Canal. This should be complete by next season.

EXPLOSION CLEARS EARTH COFFER-DAM IN FRONT OF OLD TROY DAM

The Memphis Bridge Accident

The account of the Memphis falsework failure given last week (page 1291) is supplemented by the two accompanying views, taken Dec. 24, the day after the accident. One more bent was carried away the next day, Dec. 25. The weight of the bottom chords of the anchor arm helps to hold the remaining bents.

The washout was in the 418-ft. half-cantilever span at the west, or Arkansas, end of the bridge, between piers 3 and 4. The 186-ft. cantilever arm projecting from pier 3 (an extension from the fixed span 2-3) was being erected; the bottom chord had been placed. No part of the suspended span was erected. The falsework under the suspended span was washed out. There was no mattress on the river bed, and the bottom scoured out. No lives were lost, but a traveler was wrecked.

The last pin of the fixed span (span 2-3) was driven Dec. 22, after rush work to get the span together before the height of the flood wave. The portals also had been placed. On account of the high water the span has not yet been entirely swung; a little load is left on the falsework to help hold it in place. The river bottom here is protected by a mattress.

The river has been exceptionally high all season, but fell to nearly normal early in December. As the records show few rises at this period, no trouble was expected; the only high water at this time of year was in 1906 and 1911. But a rise commenced suddenly on Dec. 18, water coming in from the Ohio River, and the stage rose rapidly until at the time of the accident the water level had reached about 28½ ft. on the Memphis gage.

The only steel lost was some stringers for the highway floor, which were being used as traveler runways on the falsework.

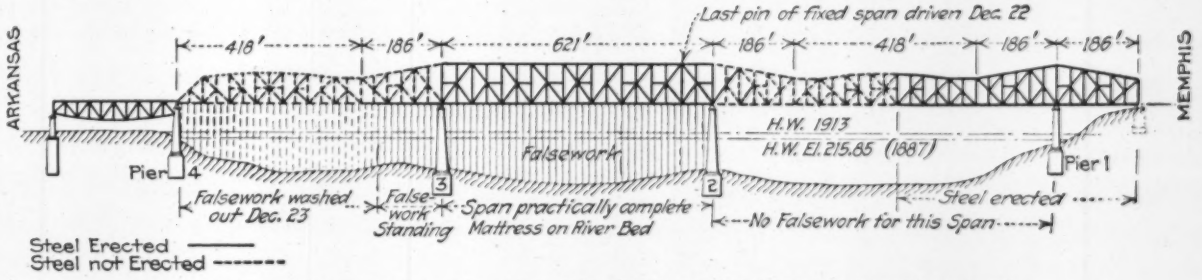
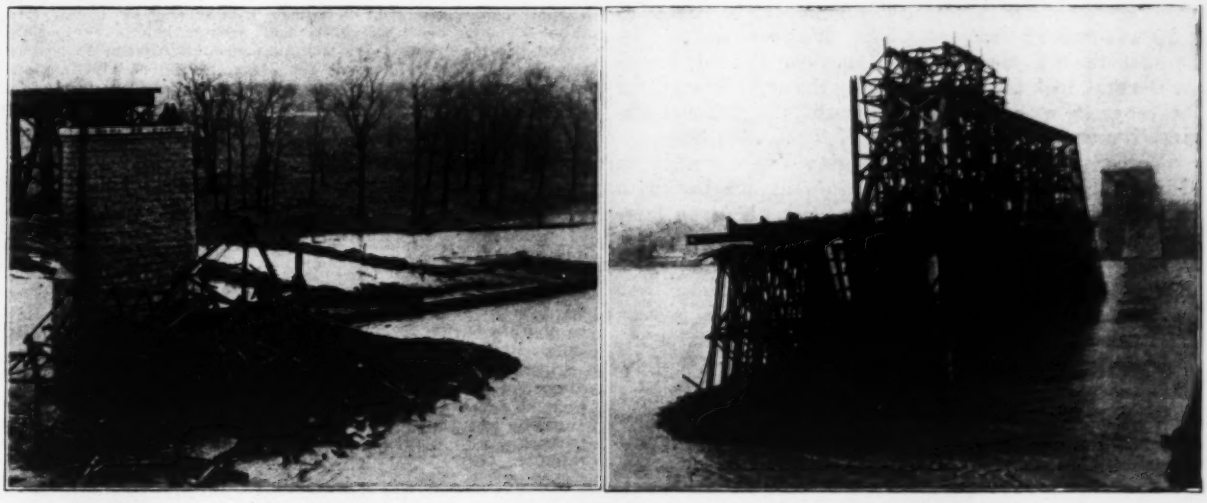


FIG. 1. DIAGRAM SHOWING ERECTION CONDITION OF NEW MEMPHIS BRIDGE

Flood Prevention at Dayton

With the first of the year the Miami Conservancy District takes permanent headquarters in a building all its own. The building has cost about $100,000 and is a gift to the district from E. A. Deeds, president of the Board of Directors of the district. It is of Bedford stone and concrete, with window and door frames of bronze. No wood enters into the construction at any point. For the present the second floor will be occupied by the offices of the directors, secretary and chief engineer, and by the files: the main floor by engineers and draftsmen; and the basement by a part of the engineering force and the appraisers, with rooms for a testing laboratory, for boring samples, for blueprints and photostat work, and by a dressing room with lockers for fieldmen.

With the formal organization of the district during the summer of 1915 the engineering work, which had been in the hands of the Morgan Engineering Co., of Memphis, Tenn., was taken over by the district, Arthur E. Morgan, president of the Morgan Engineering Co., continuing as Chief Engineer. A consulting board consisting

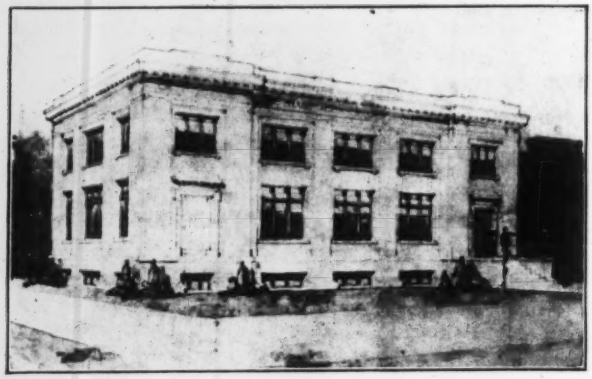


FIGS. 2 AND 3. SHORE END AND RIVER END OF OPENING 3-4 IN WHICH FALSEWORK WAS WASHED OUT

Fig. 2 shows pier 3, approach span at right. The half-cantilever between piers 3 and 4 will have its suspended span resting on pier 4. Fig. 3 shows pier 3 and the fixed span 2-3, with falsework and bottom chord of the cantilever arm in the foreground

of D. W. Mead and John W. Alvord, of Chicago, and Gen. H. M. Chittenden, of Seattle, Wash., has passed on the principal features of the plans. The announcement recently that General Chittenden was to give his entire time to the work was an error. Charles H. Paul, formerly construction engineer on the Arrowrock dam, recently joined the engineering force of the district as engineer on dam construction.

The dams for the retarding basins, exclusive of land, will cost between $7,000,000 and $8,000,000; the cost of channel improvement will be between $4,000,000 and $5,000,000; and the cost of railroad adjustments about $2,000,000. The entire cost of the project will be between $17,000,000 and $20,000,000. Mr. Morgan states that the "official plan" will be presented to the board in February. When this is completed, attention will be directed to issuing in the form of a report the results of the hydraulic and other investigations made during the preparation of the plan.



HEADQUARTERS BUILDING, MIAMI CONSERVANCY DISTRICT, DAYTON, OHIO

Engineering at Pan-American Scientific Congress

There was convened at Washington on Dec. 27 the second Pan-American Scientific Congress. (The first congress was held at Santiago, Chile, nine years ago.) The Pan-American Union through Hon. John Barrett, Director General, took the lead in making the arrangements for the congress with the coöperation of the national engineering and scientific societies of the United States.

The congress was divided into nine different sections, of which Section V was devoted to Engineering, with Gen. William H. Bixby as chairman. Other sections, however, dealt with matters of interest to engineers. Section VII, Mining, Metallurgy, Geology and Chemistry, had numerous papers on mining engineering; there were papers relating to irrigation and water-power in Section III. "Conservation of Resources" and important papers on engineering education were read in Section IV, Education.

Not only was the congress subdivided into nine separate sections, but each section was further divided into subsections, Engineering, for example, being subdivided into six subsections: (1) Civil, (2) Marine, (3) Electricity, (4) Reclamation, Sewerage and Water-Supply, (5) Mechanical, (6) Nomenclature, Standards, Mathematics, Surveys, Public Parks and Buildings.

The natural result of this minute subdivision was that most section meetings had very slim attendance. In fact the meetings of various scientific societies which were held in Washington at the same time—the American Economic Association, the American Sociological Society, the American Civic Association and a number of others—attracted much larger audiences than the Scientific Congress meetings.

The opening general sessions of the congress were made noteworthy by the participation of the Cabinet officers and other prominent Government officials and the members of the diplomatic staffs of South American countries. The Chilean ambassador, Eduardo Suarez, was president of the congress.

Over 75 papers were on the program for presentation in the Engineering Section alone. Most of them gave a general summary of engineering practice in the field covered by their subject. One paper, however, was of such timely interest and importance that it deserves special mention here. This paper, by Rome S. Brown, of Minneapolis, was entitled "Laws and Regulations Governing the Use of Water in Pan-American Countries." The author reviewed the origin and development of water laws in the United States and in Spanish-American countries and set forth clearly the legal obstacles which hinder the investment of capital in water-power development. The following is the concluding paragraph of his paper:

> There are millions of dollars of capital in the hands of American financiers ready for investment in water-power developments not only in the United States, but in all of the Pan-American republics, but which are withheld from such investments because of the financial obstacles presented in these various countries through an almost universal absence of a legislative policy which will allow such investments to be made with reasonable safety.

The Nitrate Industry

In a paper on the Chilean nitrate industry, by Enrique Cuevas, of Chile, the following figures were given, which are of interest in connection with projects for manufacturing nitrates in the United States by hydro-electric power:

> According to the latest official report of the inspector general of the nitrate deposits, the zone of nitrate-bearing ground comprises 200,000 square kilometers, of which less than 3% has been surveyed and prospected. In this surveyed portion alone there remains enough nitrate to supply the world for an additional 100 years, measured at the present rate of production. These calculations have been made on an extremely conservative basis and take no account whatever of known areas of low-grade material.
>
> The industry is divided into two principal operations: (1) Mining and transportation of the crude material, and (2) production of nitrate. Methods employed in both branches leave room for improvement. Under present practice, considering average value of raw material, wages, etc., the cost of producing commercial nitrate is estimated at 50c. per quintal (101.44 lb.)
>
> Trade specifications require a content of nitrate of sodium equal to at least 95%. This is used as a fertilizer. The refined product, with over 96% nitrate of sodium, is applied to manufacturing uses, such as glass, explosives, fusing mixtures, nitric acid, etc.
>
> There can be no competition between Chilean nitrate and atmospheric nitrate under any conditions which may be foreseen at present. The deposits of the great Pampa Salitrera (nitrate plain) are too vast and the business of manufacturing and marketing the material is too well systematized.

Papers discussing the achievements and possibilities in the artificial production of nitrates were read by Prof. G. A. Roush, of Lehigh University, and Thomas H. Norton, of the Department of Commerce.

The number of delagates from South and Central America present at the congress was about 200. Those registered from the United States numbered about 1,000. A large proportion of them came to Washington chiefly to attend the other scientific societies which held sessions at the same time as the congress. The main purpose of the congress, however, the promotion of cordial relations with leading men of the other American republics, was successfully accomplished.

Congress has appropriated $70,000 for the expenses of the congress and the entertainment of the foreign delegates. After the close of the congress on Jan. 8, these visitors are to be taken on an excursion, visiting Baltimore, Philadelphia, New York, Boston, Niagara Falls and Pittsburgh.

University Solves Ohio Flood Problem in New Way

The Ohio State University *News* for Dec. 17, 1915, has the following publicity notice:

The conservancy engineers who have been studying flood protection on the Scioto and Olentangy Rivers have abandoned the plan of securing protection by merely widening the channels, as was at first proposed, which would secure restricted benefits only locally at places along the streams where such widening would be done. They now contemplate a project which will secure more widespread benefits.

In this connection a novel plan is proposed in a bulletin just issued by the Ohio State University. This plan proposes to turn Scioto flood waters into Lake Erie. The surprising part of this project is that it proposes to give relief to the Sandusky Valley in this way. Also the area west of Marion and the Scioto and Olentangy valleys are protected. Army engineers have, however, tentatively approved the general plan as being able to accomplish these seemingly conflicting ends. Not only flood protection, but many other benefits, such as water-supplies for all cities in both valleys, water power, a navigable route 35 mi. long with lakes, are to be secured without added expense, it is claimed.

Since perpetual stream regulation has now been entered upon at several places in the state by conservancy districts under the Vonderheide law, the question of what to do with our streams in the 50 or 100 yr. between great floods is now very pertinent.

The bulletin referred to discusses, in the first seven chapters, the above questions from a state-wide standpoint, and then proceeds with a specific application to the treatment of the Sandusky and Scioto Rivers. The remarkable conclusion is reached that it will be cheaper and more beneficial to protect both the Sandusky and Scioto valleys rather than just one.

The bulletin is No. 15 of the College of Engineering. Five hundred copies have been ordered by the city engineer of Columbus for local distribution. A limited edition is for free distribution by the university.

The Cleveland & Youngstown R.R. freight terminal at Cleveland, Ohio, authorized by the voters on Nov. 3, 1915, is to be started soon. Bids are to be taken for the initial structure some time this month.

An Interesting Lawsuit has been begun in New York City, involving the status of the only woman member of the American Society of Civil Engineers. Nora Stanton Blatch, listed in the 1915 membership list of the society as an assistant engineer, Public Service Commission, First District, New York, has made an application for a writ of mandamus requiring the society to admit her to associate membership. She is a graduate in civil engineering of Cornell University and has been a junior member of the society since 1906. Having become 32 yr. of age, she has been dropped from the membership rolls in accordance with the constitution of the society, which prohibits membership in this grade of persons over 32 yr. old.

A National Conference on Immigration and Americanization is to meet in Philadelphia on Jan. 19 and 20. The object of the conference is to discuss means of systematically inducing alien workmen to take out naturalization papers and become citizens of the United States. The passage of laws concerning the employment of alien labor on public works makes this conference of especial interest to engineers and contractors. Among the members of the general committee in charge of the conference are Messrs. R. T. Crane, Coleman du Pont, Thomas A. Edison, Howard Elliott and Samuel Rea. Frank Trumbull, 18 West 34th St., New York City, is chairman of the committee.

Stray Current Worked a Signal Relay on the Nashville, Chattanooga & St. Louis Ry., Dec. 23, 1915, causing a wreck between a fast freight and a passenger train at a point 91 mi. south of Nashville. An engineer, a fireman and 13 negroes were killed. The Interstate Commerce Commission sent two signal engineers and two inspectors to investigate the accident. The railway cleared the track and had the injured under care within 30 min. after the collision. The statement of the railroad is as follows:

The train movement in the territory in which the wreck occurred is made under the controlled manual-block system. Signals cannot be given to opposing trains without cooperation of the operators at each end of the block. Investigation shows that the signal to the south-bound freight train was not given by the operator but was caused by current from some outside source picking up the signal relay. The signal wires of the railroad company and the Western Union wires were on the same pole, the signal wires being underneath those of the Western Union. It is reported that Western Union forces were working on their wires in that vicinity Thursday afternoon.

Federal Appropriations for Public Works recommended to Congress by the executive departments for the fiscal year ending June 30, 1917, contain a number of items of interest to engineers. The Treasury Department asks for $10,054,000 for the purchase of sites and construction of public buildings and $450,000 for the purchase of mechanical equipment for existing buildings. These estimates cover work to be done on projects previously authorized by Congress. The War Department asks for $25,791,972 in connection with the fortifications and defenses of the United States and $45,844,458 for carrying on work on existing river and harbor projects. The Panama Canal estimate is $19,301,369, including new work in addition to operation and maintenance, with a further sum of $8,234,100 for fortifications. The Interior Department's principal items are $8,938,000 for the Reclamation Service and $1,000,000 for irrigation on Indian reservations. The Department of Commerce calls for $2,252,300, which includes public work for the Lighthouse Service and for the Bureau of Standards. The estimate for public works under the Navy Department is $3,959,000, not including any appropriation for construction of vessels.

Typhoid Fever in the United States Army in 1914 reached a total of only seven cases and three deaths among a mean strength of 98,649 officers and enlisted men stationed in the United States and elsewhere, according to the annual report of the surgeon-general, United States Army. Only two of the seven cases had received the complete course of vaccine immunization and neither of these died. Voluntary immunization was begun in 1909, but the treatment was not made compulsory for the entire army until 1911. The army typhoid case and death rate per 1,000 from 1905 to 1914 has been as follows:

Year	Case Rate	Death Rate	Year	Case Rate	Death Rate
1905*	3.57	0.30	1910	2.37	0.17
1906*	5.66	0.28	1911	0.76	0.10
1907*	3.53	0.19	1912	0.29	0.05
1908	3.02	0.26	1913	0.05	0.00
1909	2.96	0.28	1914	0.11	0.05

*Officers not included.

Among troops scattered along the American border and in the large camps in Texas there has not been a case of typhoid fever since June 4, 1912.

Promotion in the United States Navy by merit instead of by seniority is recommended in the annual report of Secretary Daniels. The objections to the long-established system of promotion by seniority have been accentuated by the action of the last Congress in abolishing the system of compulsory retirement of officers which has been in force since 1899. This "plucking" system, so called, relieved to some extent the stagnation in the lower grades and the very short term of service in the higher grades, but involved so many other serious objections that its repeal in Congress was by almost unanimous vote. If the efficiency of the navy is to be maintained, something different from the old seniority system of promotion must be established. Secretary Daniels recommends that in order to avoid promotion by political influence the recommendations for promotion should be made by the naval officers themselves. He says: "A system of promotion based practically upon the vote of the officers senior to the grade under consideration would unquestionably select approximately the most efficient officer in that grade."

PERSONALS

Mr. Walter H. Kestin has been appointed Sales Manager of the Concrete Machinery Department of the Northwestern Steel and Iron Works, of Eau Claire, Wis.

Mr. T. L. Hughes has been appointed District Sales Manager, at New Castle, Penn., of the Lehigh Portland Cement Co. He succeeds Mr. P. A. Jandernal, who recently resigned.

Mr. George J. Adamson, formerly Special Engineer of Construction of the Union Pacific R.R. at Omaha, Neb., has been made Division Engineer of the Wyoming division of the road, with headquarters at Cheyenne, Wyo.

Mr. Trygve D. Yensen, First Assistant in the engineering experiment station, Department of Electrical Engineering, University of Illinois, has been promoted to be Research Assistant Professor of Electrical Engineering.

Mr. George F. Maitland, M. Am. Soc. C. E., Division Engineer, Maintenance-of-Way, of the Union Pacific R.R., at Cheyenne, Wyo., has been made Division Engineer of the Kansas Division, with headquarters at Kansas City, Mo.

Mr. John B. Dickson, General Superintendent of the Erie R.R., at Cleveland, has been made General Manager. He succeeds Mr. R. S. Parson, who has been promoted to be Assistant to the President and Chief Engineer with headquarters in New York, as noted elsewhere.

Mr. Arthur E. Sweet, of Topeka, Kan., General Manager of the Chicago, Rock Island & Pacific Ry., has been elected Vice-President and General Manager in charge of operation of the Denver & Rio Grande R.R. He will succeed Mr. F. L. Brown, who recently resigned.

Mr. Oscar Erlandsen, M. Am. Soc. C. E., President of the Metropolis Engineering Co., of Jamaica, Long Island, has been elected Chairman of the Borough Planning Committee by the Board of Directors of the Chamber of Commerce of the Borough of Queens, New York City.

Mr. Frederick A. Benz, formerly Assistant Engineer of the Buffalo, Rochester & Pittsburgh Ry., has been promoted to the rank of Division Engineer of Divisions 1 and 2 and the Erie division, with headquarters in Rochester. He has been in the service of the road since 1906.

Mr. O. S. Beyer, Jr., Jun. Am. Soc. M. E., General Foreman of the Chicago, Rock Island & Pacific Ry. at Horton, Kan., has been appointed First Assistant in the engineering experiment station in the Department of Railway Engineering of the University of Illinois, Urbana, Ill.

Mr. J. H. Fishback, for a number of years Assistant to the Secretary of the Interstate Commerce Commission, has resigned to enter upon the practice of interstate-commerce law. He has been chairman of the important board of reference of the commission and also a member of the Board of Suspensions.

Mr. W. T. Lechlider, Superintendent of the Cleveland division of the Baltimore & Ohio R.R., has resigned to accept an appointment as Vice-President and General Manager of the River Terminal Railway Co., which operates the railroad system at the Corrigan & McKinney Co.'s plant in the upper Cuyahoga Valley in Ohio.

Mr. Virgil G. Marani, M. Am. Soc. C. E., of Marani & Moore, Consulting Engineers, 1900 Euclid Building, Cleveland, Ohio, has opened offices at 205 Monroe St., Chicago, as Consulting Engineer for the United States Gypsum Co. He will retain his interest in consulting work in Cleveland. Mr. Marani was formerly Building Inspector of the City of Cleveland.

Mr. Alfred H. Hartman, M. Am. Soc. C. E., Division Engineer of the Baltimore Sewerage Commission, will carry to completion the work of the combined storm-water, low-level and high-level sanitary divisions. Mr. Hartman has been connected with the Baltimore work since 1906, serving first as Designing Engineer and for the past six years as Division Engineer.

Messrs. Virgil D. Allen, Building Commissioner of the City of Cleveland, Ohio, for the past four years, and Irwin S. Osborn, Assoc. M. Am. Soc. C. E., Consulting Engineer, Toronto, Can., have formed the Allen-Osborn Co., Consulting Engineers, with offices at 955 Rose Building, Cleveland, Ohio. The company will specialize in factory and mill-building construction, municipal problems and garbage reduction.

Messrs. John Cotton Dana, Librarian, and Harland Bartholomew, Secretary, have tendered their resignations to the City Plan Commission of Newark, N. J. Mr. Dana, it is stated, gave as his reason the fact that he had a great many other responsibilities and that his health would not permit him to continue his work as librarian. Mr. Bartholomew is said to have been offered a similar position with a considerably larger salary by another city.

Mr. E. L. Dunbar has resigned as Director of the Water-Works Department of Bay City, Mich., after 44 years' service. In view of his long and meritorious service, the Board of Aldermen of that city has passed the following resolution:

Resolved, That we do hereby voice our appreciation and sincere thanks for the many years of faithful and untiring efforts for the advancement and improvement of the department; that we regret to lose his valuable services and do hereby extend our hearty wishes that he will enjoy life.

Mayor Louis P. Fuhrmann, of Buffalo, N. Y., which went under the commission-plan of city government on Jan. 1, has announced the appointments of the bureau heads. Only two new appointments are made: Mr. William F. Schwartz, president of a street-sweeping machine company, has been appointed Street Commissioner; and Mr. Thomas W. Kennedy has been appointed Water Commissioner. Mr. George H. Norton has been reappointed Deputy Commissioner of Public Works and City Engineer, and his salary has been increased from $4,000 to $5,000 per annum.

Messrs. J. F. Poland and R. E. Miller, Jun. Am. Soc. M. E., have formed a partnership under the name of the Poland-Miller Co., with offices in the Erie Building, Cleveland, Ohio. Mr. Poland was formerly associated with the Brown Hoisting Machinery Co. as a civil and structural engineer, and in the same capacity with the J. W. Frazier Co. Mr. Miller has recently been associated with the J. W. Frazier Co. as a mechanical and electrical engineer. The firm has been organized to conduct a general engineering business, specializing in the design and inspection of coal, coke and ore handling machinery, docks and industrial plants.

Lieut.-Col. Edward Jadwin, Corps of Engineers, U. S. A., has been transferred to Pittsburgh, where he will have charge of the River and Harbor District of Pittsburgh and of the improvement work in connection with the Monongahela, Alleghany and upper Ohio rivers. Colonel Jadwin has been attached to the office of the Chief of Engineers at Washington. He graduated from West Point in 1890 and has had various details, including that of Division Engineer on the Panama Canal construction work. He will be succeeded in Washington by Lieut.-Col. Charles Keller, Corps of Engineers, M. Am. Soc. C. E., who is now in charge of the River and Harbor work at Mobile, Ala.

Mr. Henry W. Hodge, M. Am. Soc. C. E., Consulting Engineer, New York City, has been appointed by Governor Whitman a member of the Public Service Commission of New York City, to succeed Commissioner Robert C. Wood, who resigned his office on Jan. 4. Mr. Hodge is one of the leading men in the bridge-engineering field, having for many years been a partner of the late Alfred P. Boller, of the firm of Boller & Hodge. He is a director of the American Society of Civil Engineers. The appointment by Governor Whitman of an engineer of ability and reputation as a member of the Public Service Commission is in accordance with the recommendations heretofore urged by the engineering societies and will be heartily commended by the profession.

Mr. Robert S. Parsons, M. Am. Soc. C. E., General Manager of the Erie R.R. at Chicago, has been appointed Assistant to the President and Chief Engineer, with headquarters at New York City. He was born in Hohokus, N. J., in 1873, and graduated from Rutgers College as a bachelor of science in 1895. He received the degree of civil engineer from Rutgers in 1900. In 1895 Mr. Parsons entered the employ of the Erie as transitman on the New York division, and since then has been consecutively Assistant Engineer of the Buffalo division, Assistant Engineer of the New York, Susquehanna & Western and the Wilkes-Barre & Eastern railroads; Division Engineer of the same roads and Engineer of Maintenance-of-Way of the Erie at Cleveland; Superintendent of the Susquehanna division of the Erie; Superintendent of the New York division of the same road and New Jersey & New York R.R., Greenwood Lake R.R. and Northern Railroad of New Jersey. In 1913 he was made Assistant General Manager of the Erie at Chicago, and later became General Manager, which position he has held up to the present time.

OBITUARY

John Andrews, one of the pioneer contractors and builders of Kansas City, Mo., died at his daughter's home in that city, Dec. 28, at 65 years of age.

Thomas F. Richardson, M. Am. Soc. C. E., former Chief Civil Engineer of J. G. White & Co., New York City, died at

his home in Rutherford, N. J., Dec. 26, 1915, aged 60 years. A biographical sketch will appear in a later issue.

Gen. Grenville M. Dodge, Chief Engineer of the Union Pacific R.R. during its construction across the Western plains, died, Jan. 3, at his home in Council Bluffs, Iowa, in his 85th year. A biographical sketch and portrait will appear in a later issue.

Calvin G. Goodrich, President of the Twin City Rapid Transit Co. and St. Paul City Railway Co., died at his home in Minneapolis, Dec. 28. He was born in Oxford, Ohio, in 1856 and educated in the schools of Minneapolis. For a short time he was engaged in the wheat commission business and in 1877 entered the service of the Minneapolis Street Railway Co. as a bookkeeper. He became successively Superintendent, General Manager and Vice-President, and in 1909 was elected President. Since 1900 he had also been President of the Duluth-Superior Traction Co., owned by the same interests.

ENGINEERING SOCIETIES

COMING MEETINGS

ALABAMA ASSOCIATION OF HIGHWAY ENGINEERS.
Jan. 8. Annual meeting in Montgomery. Secy., J. B. Converse, City Engineer, Selma, Ala.

LOUSIANA ENGINEERING SOCIETY.
Jan. 8. Annual meeting in New Orleans. Secy., W. T. Hogg, State Museum Bldg., New Orleans, La.

WESTERN SOCIETY OF ENGINEERS.
Jan. 12. Annual meeting in Chicago. Acting Secy., E. N. Layfield, 1735 Monadnock Block, Chicago, Ill.

ENGINEERS' SOCIETY OF PENNSYLVANIA.
Jan. 14. Annual meeting in Harrisburg. Secy., E. R. Dasher, 30 So. Front St., Harrisburg, Penn.

CENTRAL RAILWAY CLUB.
Jan. 14. Annual meeeting in Buffalo, N. Y. Secy., H. D. Vought, 95 Liberty St., New York City.

WESTERN PAVING BRICK MANUFACTURERS' ASSOCIATION.
Jan. 14-15. Annual meeting in Kansas City, Mo. Secy., G. W. Thurston, 416 Dwight Bldg., Kansas City, Mo.

MONTANA INSTITUTE OF MUNICIPAL ENGINEERS.
Jan. 17-19. Annual meeting at Billings, Mont. Secy., C. C. Widener, Bozeman, Mont.

ENGINEERS' SOCIETY OF WESTERN PENNSYLVANIA.
Jan. 18. Annual meeting in Pittsburgh. Secy., E. K. Hiles, 2511 Oliver Building, Pittsburgh, Penn.

KANSAS ENGINEERING SOCIETY.
Jan. 18-19. Annual meeting at Kansas State Agricultural College. Secy., C. A. Foster, Topeka, Kan.

AMERICAN WOOD-PRESERVERS' ASSOCIATION.
Jan. 18-20. Annual convention in Chicago. Secy., F. J. Angier, Mount Royal Station, Baltimore, Md.

THE FRANKLIN INSTITUTE.
Jan. 19. Annual meeting in Philadelphia. Secy., R. B. Owens, Philadelphia, Penn.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
Jan. 19-20. Annual meeting in New York City. Secy., C. W. Hunt.

NATIONAL DRAINAGE CONGRESS.
Jan. 19-21. Annual meeting, Cairo, Ill. Vice-Pres., Edmund T. Perkins, First National Bank Building, Chicago, Ill.

NATIONAL SOCIETY FOR THE PROMOTION OF INDUSTRIAL EDUCATION.
Jan. 20-22. Annual convention in Minneapolis. Secy., Alvin E. Dodd, 140 West 42nd St., New York City.

ILLINOIS CHAPTER: AMERICAN WATER-WORKS ASSOCIATION.
Jan. 25-27. Annual meeting, University of Illinois, Urbana. Secy., Dr. Edward Bartow, Urbana, Ill.

ILLINOIS SOCIETY OF ENGINEERS AND SURVEYORS.
Jan. 26-28. Annual meeting, University of Illinois, Urbana, Ill. Secy., E. E. R. Tratman, Wheaton, Ill.

WISCONSIN HIGHWAY COMMISSIONERS' ASSOCIATION.
Jan. 31. Annual meeting in Madison. Secy., G. H. Mainwaring, Gotham, Wis.

INDIANA SANITARY AND WATER-SUPPLY ASSOCIATION.
Feb. 2-5. Annual meeting in Indianapolis. Asst. Secy., W. F. King, Indianapolis, Ind.

INDIANA ENGINEERING SOCIETY.
Feb. 3-5. Annual meeting in Indianapolis. Secy., Charles Brossmann, 1616 Merchants Bank Bldg., Indianapolis, Ind.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Feb. 4. Mid-winter meeting in Chicago. Secy., E. B. Burritt, 8 W. 40th St., New York City.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
Feb. 8-9. Mid-winter meeting in New York City. Secy., F. L. Hutchinson, 29 W. 39th St., New York City.

OHIO ENGINEERING SOCIETY.
Feb. 9-11. Annual meeting in Columbus. Secy., John Laylin, Norwalk, Ohio.

NINTH CHICAGO CEMENT SHOW.
Feb. 12-19. First Infantry Armory and Coliseum. Secy., R. F. Hall, 208 South La Salle St., Chicago, Ill.

CONNECTICUT SOCIETY OF CIVIL ENGINEERS.
Feb. 14-15. Annual meeting at New Haven. Secy., J. Frederick Jackson, Chamber of Commerce Bldg., New Haven, Conn.

AMERICAN CONCRETE INSTITUTE.
Feb. 14-17. Annual convention in Chicago. Secy., John M. Goodell, 1418 Walnut St., Philadelphia, Penn.

AMERICAN INSTITUTE OF MINING ENGINEERS.
Feb. 14-17. Annual meeting in New York City. Secy., Bradley Stoughton, 29 W. 39th St., New York City.

ASSOCIATION OF ONTARIO LAND SURVEYORS.
Feb. 15. Annual meeting in Toronto, Can. Secy., L. V. Rorke, Parliament Building, Toronto, Ont.

SECOND NATIONAL CONFERENCE ON CONCRETE ROAD BUILDING.
Feb. 15-18. At Chicago. Secy., J. P. Beck, 208 South La Salle St., Chicago, Ill.

IOWA ENGINEERING SOCIETY.
Feb. 16-18. Annual meeting at Des Moines. Secy., J. H. Dunlap, Iowa City, Iowa.

NEW ENGLAND ASSOCIATION OF GAS ENGINEERS.
Feb. 16. Annual meeting in Boston. Secy., N. W. Gifford, 38 Central Sq., East Boston, Mass.

AMERICAN CONCRETE PIPE ASSOCIATION.
Feb. 17-18. Annual convention in Chicago. Secy., E. S. Hanson, 538 South Clark St., Chicago, Ill.

AMERICAN ROAD BUILDERS' ASSOCIATION.
Feb. 22-25. Annual convention in Pittsburgh. Secy., E. L. Powers, 150 Nassau St., New York City.

The New England Water-Works Association holds its annual meeting on Jan. 12 in Boston at the Hotel Brunswick.

The Michigan Engineering Society will hold its 37th annual convention at Grand Rapids Jan. 18 to 20. The secretary is S. J. Hoexter, Ann Arbor, Mich.

The Arkansas Association of Surveyors and Civil Engineers held its seventh annual convention in Little Rock, Dec. 30 to 31. The secretary is J. O. Jones, of Dardanelle.

National Electric Light Association—The 39th convention will be held during the week beginning May 22, 1916, in Chicago. The headquarters will be at the Congress and Auditorium hotels, where it is proposed all meetings shall be held as well as the annual exhibit. The secretary is T. C. Martin, 29 W. 39th St., New York City.

The Society of Automobile Engineers holds its annual banquet at Hotel Plaza, New York City, on Jan. 6. The society will be addressed on that occasion by Josephus Daniels and Major-General Leonard Wood, U. S. A., on "Army Transport Problems." On Jan. 5 the Standards Committee reports, while on Jan. 6 there will be addresses and papers, both in the morning and in the afternoon.

The Alabama Association of Highway Engineers will hold its annual meeting at Montgomery on Jan. 8. Subjects to be discussed are: "Should Engineers be Licensed to Practice Their Profession"; "What Relation Has County Engineer to Board of Revenue or to County Commissioners"; "Should Counties Pay Their Highway Engineers a Specified Salary or Should They Be Paid for Engineering Service on a Percentage Basis for Work Done?"

American Civic Association—The eleventh annual convention of the association was held in Washington, D. C., in conjunction with the second Pan-American Scientific Congress, Dec. 28 to 31. The program was divided into nine sections, which included City Planning, Better Industrial Housing, Planning for a Small Town, and Country Planning. The offices of the American Civic Association are at 914 Union Trust Building, Washington, D. C.

American Civic Association—The annual meeting was held at Washington, D. C., Dec. 28 to 31, and was largely devoted to city and village planning. A session was taken up with the development of the national parks and another with the use of schools as community centers. In the election of officers, Miss Margaret Wilson, daughter of the President, and Hon. Geo. B. McAneny, of New York, were chosen Vice-Presidents. J. Horace McFarland, of Harrisburg, Penn., was re-elected President.

The Associated Technical Men, an organization devoted to the professional interests of technical men, will hold a meeting and dinner at the Morrison Hotel, Chicago, on Jan. 18. A. J. Hammond, principal assistant engineer, Chicago Union Station Co., will speak on "Engineers as Expert Witnesses." A. E. Gibson is general secretary, with offices at 1215 Monadnock Block, Chicago. A. M. Wolf is editor of the "Technician," a monthly bulletin the purpose of which is to distribute information "regarding the present unsatisfactory professional status of the technical profession, and means by which it can gain the proper recognition of the other learned professions."

American Society of Agricultural Engineers—The ninth annual meeting was held at the Sherman Hotel, Chicago, Dec. 28-30. One session was devoted to modern farm conveniences, such as water supply, sewerage, lighting and heating; this included a talk by Burton J. Ashley on small sewage disposal plants. Another session was devoted to farm tractors, their design and their economic aspects. One speaker considered them as substitutes for horses, and thought the weight should not exceed 3,000 lb. for a tractor that can be used for general purposes. Other speakers thought the tractor could not economically replace the horse for all the odds and ends of farm work. In a talk on the problems of agricultural engineering research, Prof. P. S. Rose (University of Wisconsin) advocated an investigation of power requirements and the proper combination of units for various purposes. At present dealers in machinery are not able to make such combinations and the result is that there are many "misfit outfits." The matter has been taken up by the gas engine manufacturers, who have already issued numerous data sheets giving extremely useful information. Prof. Rose proposed to have such work done by the coöperation of manufacturers, the laboratories of engineering colleges, and the Department of Agriculture (through its Bureau of Rural Engineering).

Appliances and Materials

Shadowless Safety Lantern

The accompanying view shows a new electric safety lantern designed by a practical railroad man and now being made by the Federal Sign System, Lake and Desplaines St., Chicago, Ill. Danger of explosion in working around gas, oil and other inflammables is prevented by a double set of strong guards inclosing the incandescent bulb. Shadows are prevented by placing the lamp beneath the battery compartment. The outer guard forms the standards on which the lantern rests when set down.

SHADOWLESS SAFETY LANTERN

* * *

New Safety Tread

A new form of safety tread for stairs, etc., has been devised by the American Mason Safety Tread Co., of Lowell, Mass. This product carries the trade name "Black Diamond." The body of the tread plate is made of asphaltum and carborundum. On top of this is a steel or brass skeleton frame with large diamond-shaped openings. The effect of traffic is to force the somewhat plastic material up through the grid, so that the tread wears down smooth. The metal frame confines the edges, and the cross-bars serve to drain the plate.

* * *

Black-Line Reproduction of Blueprints

The process of reproducing drawings by inked impressions from an etched plate of gelatin or paste has been perfected by the Rexim Co., 125 S. 11th St., Philadelphia, which has placed on the market sets of operating apparatus.

The following items comprise a typical outfit: A table 50x76 in. with a ground-glass top, a double boiler for heating the transfer paste, an 80-in. gutter for the table, two ink rollers, 2 lb. of "Eureka" process ink, an ink knife, a paste knife, a pan, a strainer, a large spoon, two ink plates, a thermometer, a pad, 50 lb. of "Eureka" process ink, an ink knife, a paste knife, a pan, a strainer, a large spoon, two ink plates, a thermometer, a pad, 50 lb. of "Eureka" paste, 1 lb. of "Eurepa" process varnish, a sponge, 1 lb. of "Eureka" powder, a pitcher, a dipper, a zinc-top stand and a hydrometer.

The paste is melted in the boiler and spread out on the glass-top table. When the surface is right for receiving it, an exposed blueprint (not washed) is laid face down on the paste, left for a few seconds and removed. The unexposed lines of the drawing on the blueprint leave slight etchings on the paste surface, which are inked with a roller, as in printing from any engraving. Prints are made in the usual way, by pressing paper upon the inked paste plate. The ink used will transfer to tracing cloth as well as to paper and may be had in different colors. By reinking the plate any reasonable number of reproductions may be taken. The paste is remelted and used over and over again. There are certain precautions to be observed, and a certain amount of skill is required, but the process itself is simple.

* * *

Graphic-Record Supplies

A large line of devices for the graphic representation of facts has been prepared and is being marketed by the Educational Exhibition Co., 26 Custom House St., Providence, R. I. There are authentic maps of the United States (1, 3 and 6 ft. wide), of the several states, counties, important cities and railway lines. For map marking there are colored-head needle-pointed pins; ring, number, flag, markable and other map pins; gummed paper letters and symbols, etc. Among the charting papers are various simple- and logarithmic-scale sheets, and curve cards in file sizes.

* * *

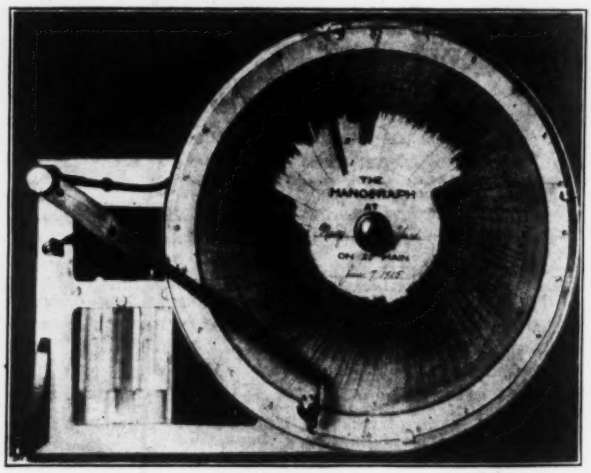
Uniform-Pressure Street Flusher

The Studebaker Co., South Bend, Ind., has placed on the market a 750-gal. street flusher which can be used for a variety of purposes. The horizontal cylindrical tank is of sheet steel, electrically welded. The chief feature of the flusher is a single-stage centrifugal pump which discharges at a pressure of 40 lb. per sq.in. The pump is driven by a four-cylinder four-cycle automobile gasoline engine at the rate of 800 r.p.m. The outfit may be used for pavement flushing, spraying trees and shrubbery, or for applying cold asphaltic or tar oils in roadbuilding. In an emergency it can be used to throw one or two streams for firefighting.

* * *

Recording Differential Pressure Gage

A new sensitive recording differential pressure gage, called the "Manograph," intended particularly for use with pitot tubes in studying leaks in water-works systems, has been devised by Paul Lanham (pitometer operator in the office of the Engineer Commissioner of the District of Columbia) and is being made and sold by him at Lanham, Md. A closed manometer U-tube with mercury is used, one leg connected to the "upstream" orifice of the pitot tube and the other to the "down-stream" side. A vertical rod, bearing a sharp platinum needle, passes through a stuffing-box and into the high-pressure compartment of the manometer. This rod is reciprocated in and out of contact with the mercury by a water motor or some equivalent. A pen arm, moving across a chart, is attached to the movable rod. The pen is not normally in contact with the chart and makes no record until an electric circuit is completed that includes an electromagnet which forces the pen against the paper. This circuit is completed when the platinum needle touches the shifting mercury level. The pen then draws a series of regular lines across the chart as shown in the view, the beginning of these lines plotting the change in differential pressure and accordingly the flow. To keep the pen from drawing on the return stroke of the contact rod, an auxiliary contact is opened by the motor. None of the moving parts derive their energy from the pressure being measured, so that the sensitiveness does not depend on the recording mechanism.



LANHAM'S DIFFERENTIAL PRESSURE RECORDER

Encyclopedia em Portuguez de Machinas para Construcções, Pagina 72

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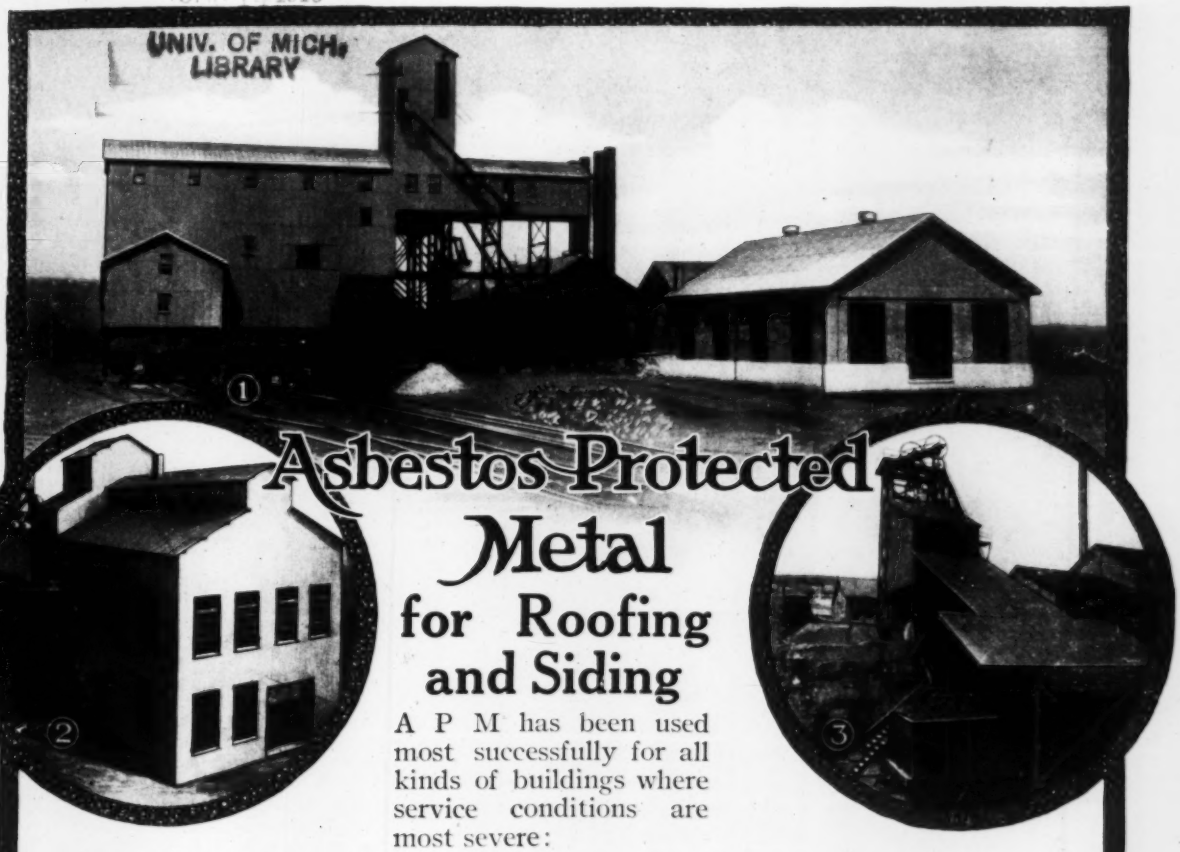

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A P M has been used most successfully for all kinds of buildings where service conditions are most severe:

First—Because of its great service-ability. Having for a base specially annealed steel sheets, completely enveloped in asphalt, onto which (while the asphalt is hot) is embedded pure asbestos felt.

Second—Because it is just as readily applied as ordinary metal sheets, but lasts indefinitely and needs no painting. It is made in various forms, (corrugated, flat, beaded, clapboard siding, etc.,) and several colors.

Above we illustrate some typical installations of A P M:

(1) Atlas Coal Company, Burgettstown, Pa. A thoroughly modern coal tipple covered with A P M.

(2) Pacific Coast Coal Company, Seattle, Wash. Here A P M is used both for the roofing and siding sheets and for window and door trim, valleys and monitor flashings.

(3) Jamison Coal & Coke Company, Greensburg, Pa. Tipples, boiler houses, and other buildings entirely covered with A P M.

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Pittsburgh
First National Bank Building

2001

Uniform Top Line Always Possible with Walls of Lackawanna Steel Sheet Piling

Approximately 860 feet of Lackawanna Steel Sheet Piling wall permanently enclosing the end rest piers of the St. John's River Swing Bridge, Florida and East Coast Ry., Jacksonville, Fla. (See last week's advertisement) had to be uniform in height, although penetration of the sheet piling to bed rock of uneven surface was necessary.

The individual sheet piles were therefore driven to refusal and the tops cut to the desired line by the oxy-acetylene process. It was found that one man was able to do the cutting at the rate of 40 to 50 lineal feet of piling wall per day.

Usually Lackawanna Steel Sheet Piling can be driven perfectly to an even top line, but if the character of the sub-soil prevents, cutting as above, always offers a practical means of conforming to requirements of appearance and utility.

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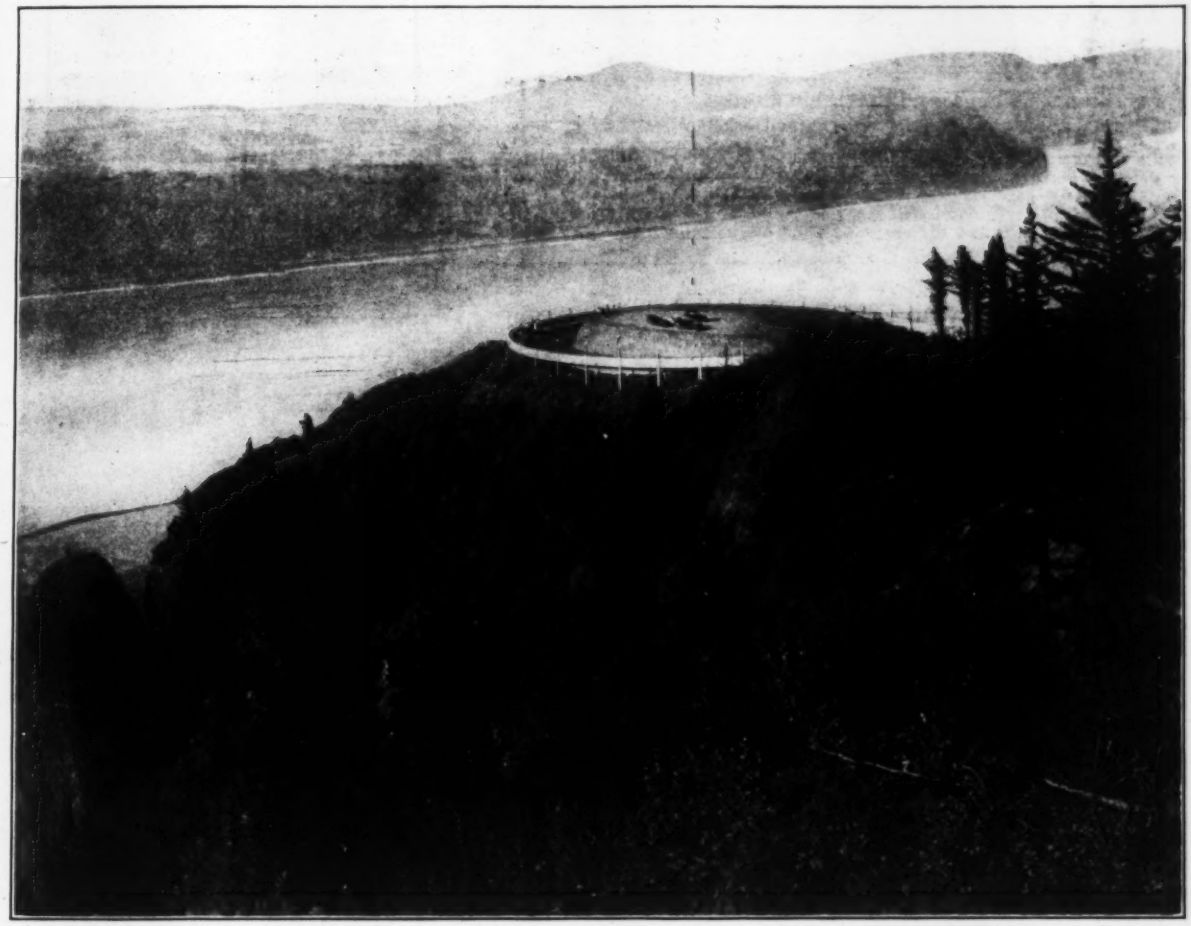
Engineering News

VOLUME 75 JANUARY 13, 1916 NUMBER 2

Multnomah County Roads in the State of Oregon

The County of Multnomah, Oregon, of which Portland is the county seat, has spent approximately $2,250,000 on highway construction during the years of 1914 and 1915, of which $1,250,000 was from a special bond issue for the hard-surfacing of 70 mi. of road. This hard-surfacing consisted of 39.5 mi. on the Columbia River Highway, of which approximately 30 mi. is new location and 9.5 mi. on old macadamized road. The remaining 30.5 mi. of the highway is on an old location, but has been realigned, and all grades exceeding 5% have been eliminated or reduced to 5% or less.

All bridges, trestles and culverts have been replaced with modern concrete construction or fills, and all important grade crossings have been eliminated by the use of viaducts or underground crossings. The Columbia River Highway has 17 bridges and viaducts, whose added length of span amounts to 3,699 ft. All of these bridges and viaducts were designed to add to the attractiveness of this now well-known scenic district. On six other paved highways radiating from the City of Portland, which have a total length of 30.5 mi., there were built three concrete bridges and viaducts, the sum of whose spans equals 394 ft.



A HIGH POINT ON MULTNOMAH COUNTY HIGHWAY, COLUMBIA RIVER VALLEY, OREGON

Approximately 60 mi. of the hard-surfacing was of asphaltic concrete (bitulithic or Warrenite) and 10 mi. was concrete pavement of 1:1½:3 mixture, one-course

construction. All of the asphaltic concrete, except 26 mi., was laid on waterbound macadam base, not less than 6 in. in depth and averaging 8 in. All of the pavements were 18 ft. in width, with a 2-ft. macadam shoulder. The contracts were awarded June 9, and 60 mi. had been finished by the last day of November.

It is interesting to note that 541 bids were received. Seven contractors received awards. There were few bids from small firms, owing to the fact that all pavement was laid under a 10-yr. maintenance bond with a 5-yr. guarantee of workmanship and materials. The cost of the paving is well within the estimated amount of $17,857 per mi.; the engineering cost is to date 2% of the construction costs, but will probably reach 2½% upon completion of the work.

The County Engineer of Multnomah County is Herbert Nann.

Data for Water-Works Design in Tropical Countries

The following extracts from the 1915 annual report of the Governor of Porto Rico are contained in the report of the Division of Sanitation of the island, F. W. Dalrymple, Sanitary Engineer, and while relating specifically to Porto Rican conditions would probably prove of value to engineers called upon to design water-works in other Spanish-American countries in the tropics or subtropics:

All possible data have been collected as to the amount of water actually used by the municipalities of the island so that they might serve as a basis for the design of new works. As few of the towns use meters (and none of them have the entire supply metered) it is difficult to get precise facts. The average per capita consumption per day ranged from 60 liters (16 gal.) in Guayama, where there is a shortage of water and the people can use it only from 8 a.m. to 4 p.m., to 130 liters (34 gal.) in San Juan and Arecibo. Both of the last-named places have meters in very general use, although they were not introduced into Arecibo until 1913; it was found that their use reduced the consumption of water from 370 liters (98 gal.) to 130 liters per capita per day.

From two years' pumping records of the San Juan works, it is found that the maximum daily use of water is liable to be 25% greater than the average, and the maximum hourly use may be one-third greater than this maximum. As the use of water is bound to increase with the more extended introduction of sewers and plumbing, an average consumption of 150 liters (40 gal.) per capita per day is believed to be a safe assumption in designing new works.

The size of the main pipe line supplying the town, however, must be sufficient to deliver water at the maximum hourly rate, which is one and one-fourth, one and one-third or one and two-thirds times the average rate; this means 250 liters per day on the given assumptions.

In addition to the above, the pipes must have sufficient capacity for fire service. In Porto Rico the number of fire streams necessary may be taken at twice the square root of the population in thousands, and the average rate of consumption of a fire stream is 1,000 liters per day. In the smaller and poorer towns it is believed these rates would be excessive and that an average per capita consumption of 100 liters per day would be ample. If x is taken as the population of a town in thousands, the number of cubic meters required under various conditions may be stated as follows:

Rate per 24 Hr.	First-Class Towns, Cu.M.	Second-Class Towns, Cu.M.
Average rate	150x	100x
Maximum rate, no fire service	250x	166x
Maximum rate with fire service	$180x + 2000\sqrt{x}$	$120x + 2000\sqrt{x}$

Difficult Falsework Erection for 242-Ft. Concrete Arch

On the line of the new California state highway that is to extend the entire length of the state from north to south, a distance of over 800 mi., there is now under construction a concrete arch bridge of 242-ft. span, which involves some difficult problems in erection.

The bridge is to cross the Pitt River near Baird, Calif., about 50 mi. south of Mount Shasta. Most of the rivers of California are either entirely dry by the end of the long rainless summer and fall or else have a flow of only trifling amount. The Pitt River, however, even at the end of the dry season is a large stream. It drains a great area of country lying east of Mount Shasta and is fed by underground streams issuing from the great lava beds of Modoc and Lassen Counties.

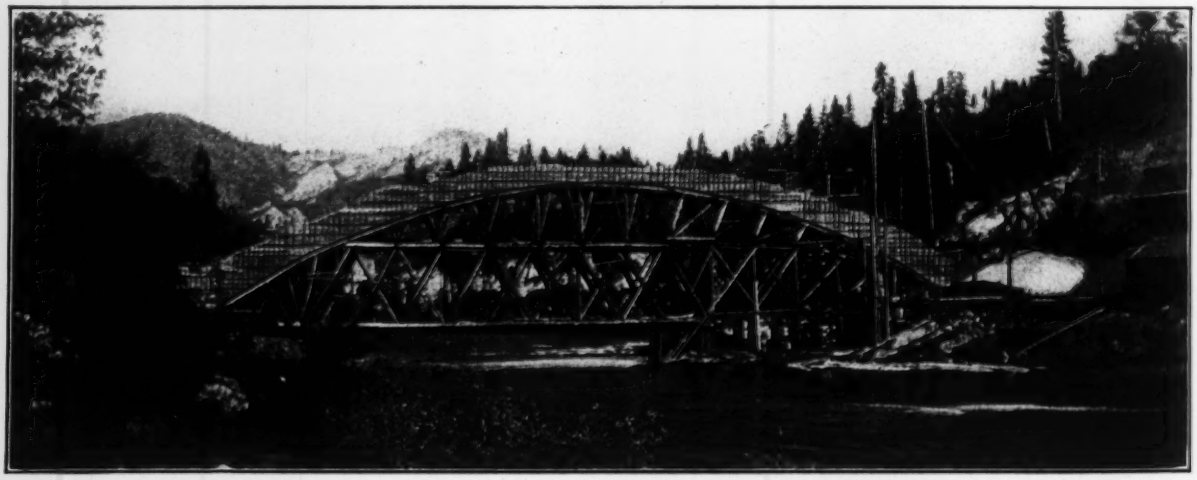


FIG. 1. HOWE TRUSS OF ROUND TIMBERS SUPPORTING FALSEWORK FOR 242-FT. CONCRETE ARCH BRIDGE

At the site of the crossing, the exposed rock on either side of the stream offers natural abutments for an arch structure, and a concrete arch was decided upon. Concrete structures are in fact used in California on permanent bridge work wherever conditions permit. The bed of the Pitt River at the bridge site, however, is solid rock; the depth of water in the channel is about 30 ft. and there is a rapid current. Driving piles or erecting trestle bents to support the falsework in the ordinary manner was not feasible.

The contract for constructing the bridge was awarded by the State Highway Commission to the Ross Construction Co., of Sacramento, Calif., and the chief engineer of the company, George G. Pollock, devised the system

of falsework for erection illustrated herewith. As shown, a Howe truss is carried across the stream, being supported on trestle bents near each shore. This truss supports the struts and lagging on which the concrete arch is built.

The chief problem, however, was how to erect the truss itself. The gorge to be crossed was too deep and narrow to permit of erecting the span on shore and shoving it out endwise over the stream as has sometimes been done. Instead, Mr. Pollock devised a plan for erecting the truss span in place.

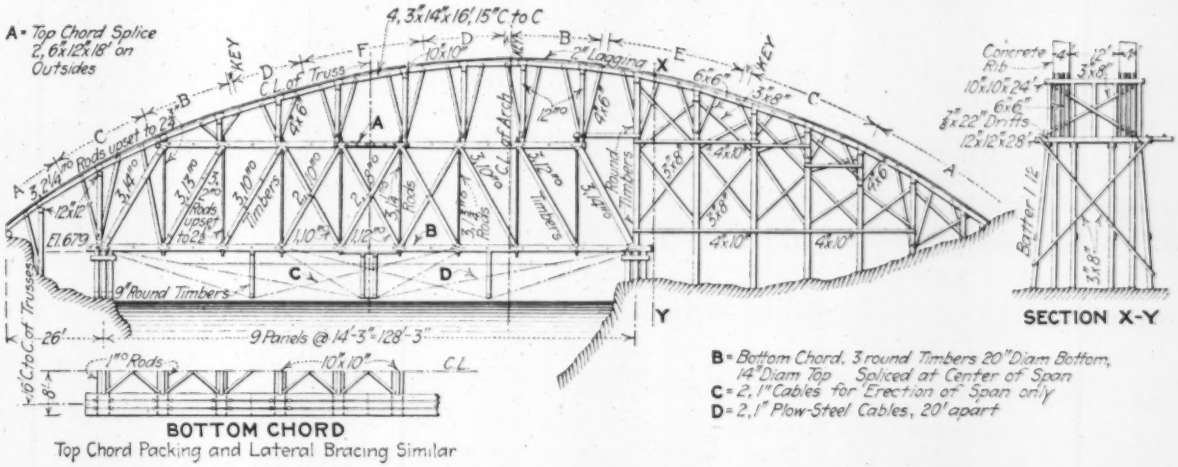


FIG. 2. FALSEWORK FOR BAIRD BRIDGE, SHOWING METHOD OF ERECTING HOWE TRUSS

The lower chord of the truss is made up of sticks of round pine timber cut near the bridge site, and long enough so that only one joint is necessary—at the center of the span. This lower chord was made up first, half on each side of the river. Trestle bents were erected to support the truss at each end and extended as far out into the stream as the depth of the water would permit. The two halves of the lower chord were then swung into place, their shore ends resting on the trestle bents and the channel ends being supported by tackle from a ginpole on one bank and from an erection traveler on the other. While held in this way the two halves of the lower chord were joined. Then struts were placed on the under side of the chord, wire cables were stretched over the struts and the lower chord of the Howe truss became temporarily the top chord of a queen-post truss.

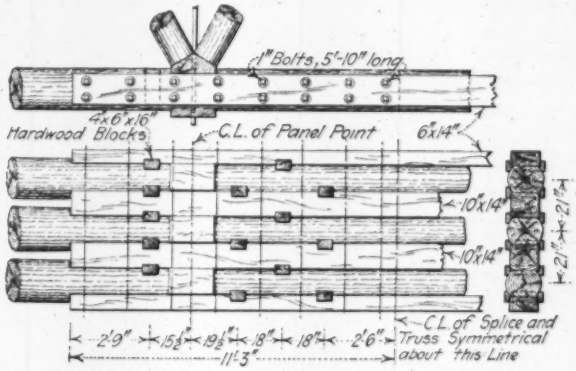


FIG. 3. DETAILS OF SPLICE FOR LOWER CHORD OF HOWE TRUSS

On this queen-post truss as a support the Howe truss was erected.

The splice used for joining the two halves of the lower chord is shown herewith. The use of the round sticks for the Howe truss gave an added margin of strength and stiffness and, what was most important, enabled the erection to be done rapidly with material available at the site, with the minimum of labor expended on it.

For the batter blocks between the ends of the braces and the lower chord, hard laurel wood was used.

The pouring of the arch ring is now in progress. This work is being done in successive sections, as indicated by the serial lettering on the accompanying drawing, and keys are to be left open at the crown and the haunches. These are to be poured after the rest of the arch ring.

Technical School Competition

In the engineering school of Ohio State University the practice is followed of having senior students, for thesis work, survey and report on projects of assistance to various communities within the state.

At Sidney three students prepared as a thesis for graduation in civil engineering, plans for eliminating railroad-grade crossings and for accompanying improvements. The city authorities paid the bare expense of doing the work, amounting to $150, which otherwise would have cost more than $1,000, for which expenditure no funds were available. The thesis thus prepared answered a double purpose: First, it supplied an original problem to be solved for graduation, and second, it furnished complete enough plans (not otherwise obtainable) so that a bond issue for carrying out the project could be submitted for vote at the November election.

Immediately after Thanksgiving six men began surveys for a combined water-power project and electric traction line between Chillicothe and Hillsboro. The proposed power development is on Paint Creek, about halfway between the two cities mentioned. The work will be done as thesis work, but is expected to have practical value, as there are scant transportation facilities in the region to be examined, and the water power to be generated could be practically all used in the proposed traction line without seeking further market.

Eight Years of Imhoff Tank Design and Operation--II*

By Karl Imhoff†

SYNOPSIS—The engineer who developed the form of two-story sewage-settling tank or modified septic tank bearing his name reviews eight years' German and American experience in design and operation of Imhoff tanks. The translator, who worked with Dr. Imhoff in Germany and was subsequently engaged on the design of Imhoff tanks for Baltimore, Md., has added a number of supplementary notes.

Tank Operation and Sludge Disposal

The operation of the settling chamber is very simple. From time to time the inclined partition walls should be cleaned by means of a broom or a squeegee. If the inclination of these walls is at least 1.2 vertical to 1 horizontal the sludge will glide off of its own accord. A greater inclination is necessary only in plants where two inclined surfaces meet each other so as to form a solid angle whose vertex or line of intersection inclines more to the horizontal than the wall surfaces themselves.

In cleaning the inside surfaces of the settling chamber it is important that the slot does not become clogged. It may happen that large-sized solids have squeezed themselves through the coarse screens and have then become engaged in the slot, thus permitting sludge accumulations at this point. The plant attendant should be taught to recognize conditions of this kind quickly and to remove them immediately. The greater portion of such suspended matters as float on the influent side of the first scum board will generally sink of their own accord after a time. Sometimes it may prove advisable to expedite their sinking by spraying upon them with a hose. Pieces of wood, cork, etc., which will not sink should be skimmed off occasionally.

In tanks where one sedimentation chamber feeds more than one sludge chamber the direction of flow of the sewage should be reversed about once every three weeks, that the sludge chambers may receive sludge uniformly. Before reversing the direction of flow it is necessary to skim off the floating matters from the surface of the settling chamber, so that they may not escape with the effluent. Such recovered floating matters should be buried. Suspended matters are sometimes found to rise to the surface behind, as well as in front of, the first scum board. This will probably be a sign either that the inner surfaces of the settling chamber have not been kept sufficiently clean or that the sludge level has risen to the slot level and hence stopped up the slot and allowed sludge to accumulate above it in the settling chamber. If the latter is the case it will be necessary to discharge sludge.

The operation of the sludge chambers, which is simple in practice, since all that is necessary is occasionally to discharge some of the sludge, is very interesting theoretically and has awakened much scientific discussion in the last few years. When the first Imhoff tanks were built in the Emscher district it was assumed, in accordance with what was universally believed at that time, that the decomposition of sludge could not proceed if flow were not maintained in the sludge chamber. However, it was considered most important of all to keep the sewage itself as fresh as possible by separating it entirely from the sludge, and so, in order to maintain the decomposition of the sludge, the sludge chambers were provided with agitation and water flushing devices. Sludge inoculation and the addition of chemicals were also considered.

The operation of the first plant built, that at Recklinghausen, showed, however, that the sludge decomposed properly without artificial stimulus. It was noted that the tank required about eight months for what came to be termed the ripening period. During these eight months it was necessary to discharge from time to time incompletely decomposed and foul-smelling sludge. This foul-smelling sludge was buried. It is significant that one year later the buried sludge was found to be still in the same foul condition. After the completion of the above-mentioned period of eight months the sludge discharged from the tank was found to be nonobnoxious and easily dryable. From that time until now, eight years later, the sludge discharged from the Recklinghausen plant has consistently maintained its good quality.

Various Phases of Decomposition of Sludge

Now that the experience of eight years is available it is clear that there are a number of different ways in which municipal sludge may decompose, just as there are various paths of decomposition which may occur in other substances, such as those which are well known in the fermentation industries. There is only one method of decomposition for sewage sludge which is technically correct. This correct method is a combination of various processes occurring simultaneously. Proper decomposition is identified by the fact that it occurs entirely without objectionable odors. The effect of such decomposition upon the sludge is that it becomes black in color, that it gives an alkaline reaction, that it possesses a not unpleasant odor like that of rubber or warm sealing wax, and that it is easily dewatered.

The period that elapses from the time the tank is placed in operation until it begins to deliver good sludge is known as the ripening period, as above mentioned. It is possible to eliminate the ripening period entirely if the tank be inoculated with a proper amount of good sludge at the beginning of operation. Often, however, even without inoculation the tank will discharge good sludge from the first. This occurred in the fairly large plant at Essen Nordwest because at the time it was placed in operation a large number of cesspools which had been in use for decades, and which therefore contained ripe sludge, were emptied into the sewer system.

*Translated by Leslie C. Frank, Sanitary Engineer, United States Public Health Service, Washington, D. C.

†Division Engineer, Emscher Federation, Essen, Germany.

If a new tank does not become inoculated immediately, either artificially or naturally, the first sludge that fills the tank will be relatively fresh. If such sludge consists largely of fresh fecal matter it will at first be yellowish-gray in color, then gray and finally black. In the earlier part of the ripening period it will happen occasionally that acid decomposition will occur. Fecal matters attributable to vegetable diet incline toward acid decomposition, whereas a meat diet tends to produce an alkaline decomposition. The process of acid decomposition is extremely undesirable because then the sludge decomposes very slowly, cannot be dried easily and has an intensely objectionable odor. In addition, sludge that is in a stage of acid decomposition tends to attain a mass specific gravity less than that of water and to rise as a body to the surface of the sludge chamber. If a tank be operated in the proper manner, which will be discussed more in detail farther on, such acid decomposition will sooner or later be replaced by the proper alkaline decomposition.

Movements of Gas and Sludge

As soon as the ripening period has been passed the sludge is alkaline, black and odorless. The gases of decomposition cause particles of the sludge to rise to the surface of the sludge chamber, where they give up their gas content and again sink to the bottom. This faculty of giving up its gas content depends in large measure upon the fact that the sludge has, through its decomposition, lost its stickiness or, more properly, its viscosity, and hence its ability to hold gases. It is because of this process of rising and falling on the part of the sludge particles that the sludge mass as a whole becomes thoroughly mixed. Fresh sludge particles are continually entering the sludge chamber through the slot and, settling to the surface of the mass of decomposing sludge, are quickly mixed more or less uniformly therewith. Consequently in a very short time they are inoculated with the proper organisms. This makes it impossible for local acid processes to occur. That is why it is never possible to find in a tank which has passed the ripening period a layer of fresh or almost fresh sludge.

From time to time decomposed sludge is discharged through the sludge pipe, whose influent end is situated at the deepest point in the sludge chamber. A tank which has once passed the ripening period will always deliver good sludge if no gross errors of operation occur. With this assumption acid decomposition is impossible. Upon the surface of the sludge chamber a relatively thin layer of scum will form which will have to be removed not oftener than once a month at the most. Previous to its removal a stream of water should be directed upon it for a short time in order to throw down any easily settleable solids.

Much emphasis should be placed upon the difference between what has just been termed the scum layer and another phenomenon, namely, foaming or floating sludge. This latter is very similar in appearance to a mass of resistant, dark-colored soap bubbles, and also distinguishes itself from the proper scum layer by its ability to rise in the gas vents far above the tank water level. It consists simply of sludge in an improper state of decomposition and possesses a sufficiently high viscosity to enable it to imprison firmly its gases. This phenomenon should never occur in a properly operated tank. Its occurrence indicates, and this will be discussed more in detail farther on, that sludge should be discharged at once.

The fact that acid decomposition sometimes takes place in tanks which have not passed the ripening period has led to the assumption, from a scientific standpoint, that a certain amount of alkalinity should be present in the sewage if a proper working of the tank is to be expected. It has been assumed that the sludge should be mixed frequently with this alkaline sewage, and that there should always be present in the sludge chamber a greater amount of sewage than sludge. It has also been believed that it might be possible to combat acid decomposition by the addition of lime to the sewage or to the sludge. I have never been able to substantiate any of these assumptions. The addition of lime has never resulted beneficially in the Emscher district.

Addition of Lime to Prevent Acidity Not Advised

It may be interesting from a scientific standpoint to note that Dr. Spillner, chemist to the Emscher Federation (Dr. Spillner has since fallen on the field of battle), proposed a theory directly contrary to the one that acid decomposition might be combated by the addition of lime. He believed it possible that the addition of lime might instead promote the acid process, as the products of the acid bacteria were inimical to their own existence, and hence neutralization by means of lime would really improve their environment. Dr. Spillner further believed that if lime were not added the acid bacteria would sooner or later cause their own destruction. He pointed out the interesting fact that acid decomposition occurred very intensely in new tanks which during the period of construction had been filled with a rich solution of milk of lime and cement, and whose walls, certainly during the first period of operation, exuded a considerable amount of alkali.

Without considering more closely these interesting bacteriological theories it will suffice to say that practical experience with dozens of plants has not substantiated the statement that the addition of lime would be beneficial. I cannot advise such addition. All plants which are known to me and in which acid decomposition has occurred have sooner or later adjusted themselves of their own accord. Even the plant at Elberfeld, in which the incoming sludge is always acid because of the presence of particular trade wastes and the sewage of which is at times very strongly acid, has finally righted itself.

To achieve proper sludge decomposition therefore depends not upon the composition of the sewage, not upon its volume, not upon the relation of sewage to sludge. The sewage may in fact be continuously acid. *Proper decomposition depends only upon the sludge*, and sooner or later the sludge itself will develop a preponderance of the proper organisms. Once good decomposition has been established it must be maintained by always providing the necessary bacteriological environment. This condition will be fulfilled by a naturally simple procedure. The following rules should be observed:

Rules for Handling Sludge Satisfactorily

(1) Sludge should be discharged from each sludge chamber once every two to six weeks. This procedure will effect the maximum utilization of the sludge-drying beds. As soon as a sludge-drying bed is free, sludge should be discharged even if thereby the level of the

sludge in the tank should be considerably depressed. The tank may not be completely emptied of sludge but must always be left with a sufficient amount to insure the maintenance of the proper decomposition process. It will be necessary therefore to stop discharging sludge from the tank if its sludge begins to show signs of deterioration, or as dictated by experience, when the sludge remaining in the tank is approximately equivalent to one month's accumulation.

(2) In general the optimum position of the sludge level is about 3 ft. below the slot level. It ought never to be allowed to rise above the lower edge of the inclined diaphragm which limits the slot. It must certainly never be allowed to rise to a level at which the slot will be closed by the sludge.

(3) The elevation of the sludge surface may be determined by lowering into the sludge chamber a very thin piece of sheet iron about 1 sq.ft. in area, held in a horizontal position. The determination of the sludge level in this manner may not be relied upon absolutely, because the transition between sludge and supernatant is often very difficult to perceive. If the sludge level is too high this fact will evidence itself by the occurrence of gas bubbles on the surface of the settling chamber vertically above the slot, even though the slot has been cleaned from above and is known not to be clogged. Another evidence of an excessively high sludge level is the occurrence of floating sludge or in extreme cases foaming sludge. In either case it will be necessary to discharge sludge immediately from the bottom of the sludge chamber. It is never necessary to remove sludge proper from the gas vents. The normal scum layer must be removed occasionally, but the floating of sludge proper or the phenomenon of foaming is absolutely preventable. These phenomena occur only if sludge is not discharged at proper intervals.

The rules which have been formulated above, referring particularly to tanks that have passed the ripening period, hold also during the ripening period, which lasts on the average about six months. Even if the sludge is still fresh, or indeed acid, it must be removed from the tank as soon as any of the above-named symptoms appear. The foul-smelling sludge that is removed may be buried, or dosed with hypochlorite and spread out in a thin layer on the ground surface. If this procedure be followed the ripening period will sooner or later come to an end and the normal operation phase begin.

If it be desired to promote the early incidence of proper decomposition it will be necessary above all to make sure that the sludge does not lie quietly at the bottom of the sludge chamber. Its proper decomposition demands, as above explained, that it be constantly stirred. It also demands that the introduction of fresh organic matter and the discharge of decomposed organic matter be uniform. Proper decomposition may therefore be promoted by artificially mixing or stirring the sludge. In particular the scum layer should be frequently agitated by a jet of water or otherwise, and the sludge at the bottom of the chamber should be agitated by a water-stirring system. As a substitute the whole body of sludge may be pumped out and returned. If it be desired to promote the action still more, inoculation may be tried. For this purpose the tank must be emptied as far as possible. If then well-decomposed sludge from a properly operating tank be introduced into this empty tank to the amount of about one week's fresh sludge accumulation, it may be assumed that the tank has been sufficiently inoculated. Inoculation may also be acquired with smaller amounts of decomposed sludge. A small amount may be mixed with fresh sludge in the proportion of 1 : 1, placed in the tank and allowed to stand one week. The mixture will by that time have become sufficiently decomposed to permit the addition of its own volume of fresh sludge. This process may then be continued until a sufficient volume of properly decomposed sludge has been acquired.

What Performance May Be Expected?

The performance of any sedimentation device must always be limited to settleable solids. The total suspended solids which the sanitary chemist is accustomed to determine are generally derived by means of laboratory filtration methods. Such a determination may not be taken as a criterion in determining the performance of any settling device. Any laboratory method which is intended to measure such performance should be one which will determine the settleable solids only. In Germany it is customary to denote as settleable such matters as will settle in an interval of two hours from the sewage contained in a one-liter glass cone. The reading is therefore volumetric.

[For most American plants the volumetric method of determining settling-chamber performance is inapplicable because the volume of solids that will settle out of a reasonable volume of sewage is not readable with the desired precision. The following method has therefore been adopted in the experiments of the United States Public Health Service upon small sewage-treatment plants: (a) Total solids are determined on the influent sewage by the evaporation method. (b) Total solids are determined on the tank effluent by the evaporation method. (c) A cylinder of the tank effluent is kept quiescent for two hours and then 50 c.c. of the supernatant carefully removed and total solids determined on it by evaporation. $(a - b) \times 100 \div (a - c)$ is then the percentage performance of the tank with regard to settleable solids.—Translator.]

It may be expected of a properly designed settling tank that it remove at least 95% of the above-denoted settleable solids. In comparison with mechanical screens such a performance is relatively very high, since the best of such screens remove only from 10% to 15% of the settleable solids and hence at the most yield a performance of only about one-sixth that of sedimentation plants.

In cases where plain sedimentation is insufficient the performance of a settling tank may be improved in various ways. It is particularly easy to combine the process of disinfection with the settling process. There has been much experience in the United States with the hypochlorite method of disinfection. A long-time experiment conducted in one of the six tanks of the sewage plant of Essen-Ruhr, Germany, showed that the process of decomposition in the sludge chamber is not disturbed by the process of disinfection applied in the settling chamber. In other words the disinfection process fortunately limits itself to the sewage and may therefore be applied without hesitation to the settling chamber of an Imhoff tank.

Chemical-precipitation methods also may be used in connection with Imhoff tanks. It has been determined

that the sludge obtained by chemical-precipitation methods will decompose properly in an Imhoff tank sludge chamber. This fact solves the long-standing problem of handling the voluminous and not easily dryable sludge obtained from chemical precipitation.

In cases where it is necessary to treat the effluent of a sedimentation plant upon sprinkling filters it is particularly important that the sewage be freed of sludge-forming matters as far as possible and at the same time retain its fresh condition.

Sewage-treatment plants should be so designed as not to cause nuisance at any stage and so that the sludge may be dried without undue difficulty. When the sludge has once been dried it may be utilized in a number of ways, for example, to fill in low ground, as a fertilizer or as a fuel. The sludge obtained from mechanical screening plants is naturally less in volume than that derived from Imhoff tanks because of the lower performance of the former.

[In cases where the performance of fine screens is sufficiently high the sludge may be treated in separate digestion tanks. Or if the screenings are removed from the screen by water jet, as at Baltimore, the wash water may be led through a special Imhoff tank with relatively small settling chamber.—Translator.]

J. Herbert Shedd--A Pioneer Municipal Engineer*

J. Herbert Shedd, who died on Nov. 27, 1915, was born May 31, 1834, in the town of Pepperell, Mass. He descended from an old colonial family whose home was in Billerica, Mass., for many generations.

He obtained his early education in the public schools of Massachusetts and at Bridgewater Academy. His preparation for the engineering profession was obtained through a three years' service in the office of Thomas Doane, in Boston. This constituted the basis of his preparation for a lifework, and we find him at once taking up railroad work of importance and then establishing his office in Boston in 1859, when he began the practice of hydraulic and sanitary engineering. He continued his Boston office in partnership with Edward Sawyer for 40 yr.

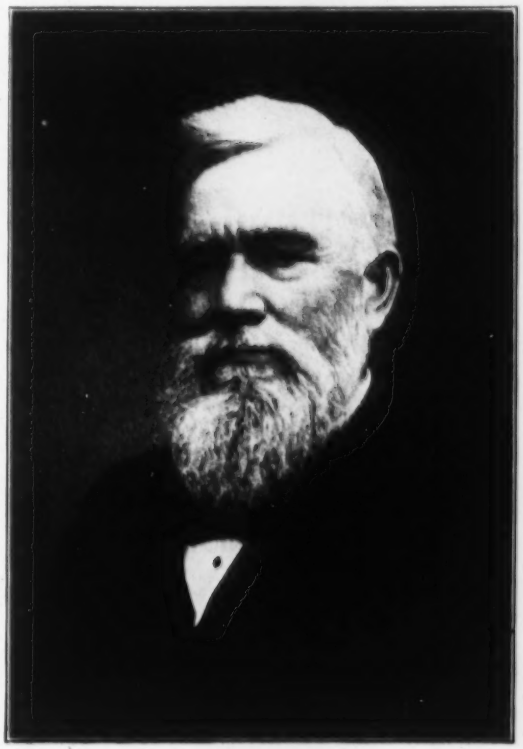
He was called upon by the City of Providence in 1866 to make investigations and recommend a plan for a water-supply and a system of water-works for that city, which he reported upon in 1868. He became a resident of Providence in 1869 and served the city in the capacity of chief engineer of the water-works until 1877. While chief engineer he reported, in 1874, upon a comprehensive plan for the sewage of Providence, which was partly carried out under his direction previous to his retirement from office.

In designing the water-works and sewerage systems of Providence, every element was studied by him fundamentally, and nothing was copied. His designs have been much referred to and have a European reputation. In connection with this work Mr. Shedd probably did more to improve the quality of American hydraulic cement than had been accomplished by any other engineer, due to the rigidity of his requirements and testing.

On completion of his engagement with the City of Providence, he opened an office in that city, where he continued to practice his profession. During the years 1878 to 1889 he was engaged in designing water-works plants and sewerage systems for numerous towns and cities.

In 1890 he was appointed city engineer of Providence, R. I., which position he held until 1896, after which he was retained as consulting engineer to the city for two years, when the sewerage system was completed.

As far back as 1880 he was retained in contested cases involving questions of damage due to the taking of water



J. Herbert Shedd.

for public supplies and on the valuation of water-works. Following his retirement from the position of consulting engineer for the City of Providence, his professional work consisted very largely of consultation with reference to various works and expert work for valuations.

In addition to his professional work Mr. Shedd enjoyed service on numerous public commissions. He was appointed commissioner for Massachusetts on the Concord and Sudbury rivers in 1860; member of the Board of Harbor Commissioners for the State of Rhode Island in 1876, of which board he was chairman continuously from its organization to his death; member of the interstate commission to fix the boundary line between the states of Rhode Island and Connecticut in 1885; member of the interstate commission to establish encroachment lines in the Pawcatuck River, 1887; member of the commission for investigating the necessity of a new bridge across the Sakonnet River, Rhode Island, and a member of the commission to build a new bridge, 1904 to 1910.

In 1904 Brown University honored him with the degree of A.M. Mr. Shedd became a member of the American Society of Civil Engineers in September, 1869, and at the Centennial Exhibition was chairman of its subcommittee on sewerage and sanitary engineering.

*Contributed by O. Perry Sarle, Civil Engineer, Providence, R. I.

Analysis of the Failure of an Earth-Fill Dam

By Guy Sterling*

SYNOPSIS—The Hatchtown dam, on the Sevier River in southern Utah, in a most inaccessible location, failed on May 25, 1914. At that time such of the particulars of the failure as were obtainable were published. Since then Mr. Sterling has investigated the design and construction of the dam and has studied the causes of its failure. The results of this investigation are given herewith as a guide for possible future work in earth-dam design.

The Hatchtown reservoir is located on the Sevier River in southern Utah, about 220 mi. south of Salt Lake City, at an elevation of about 7,000 ft. At this site in 1900 a private irrigation company built a small earth dam with a lime-mortar culvert. Soon after its construction this dam started to leak and subsequently was washed out, but reconstruction was immediately started and carried out. In May, 1901, this second dam had been built to a height of 40 ft. and provided, near the middle of its length, with a timber spillway 4 ft. deep and 20 ft. wide, to discharge on the downstream face of the rock fill, which is reported to have withstood a flow corresponding to 1 ft. in depth in the spillway. At this juncture a rising stream, due to the usual spring floods combined with high winds and inadequate spillway capacity, overtopped the dam and, except part of the culvert, carried away nearly the entire structure, although strenuous efforts were made to save the work by opening the culvert gates to full height and raising the crest of the dam. After this failure no further attempts were made by local interests to rebuild the dam.

The next dam to occupy the Hatchtown site and the one to which this article is chiefly devoted, was built by authority of the Utah State Board of Land Commissioners and under the general supervision of the State Engineer, in connection with a state irrigation project embracing about 6,000 acres of land lying below and from 12 to 20 mi. north of the site.

In the summer of 1906, soon after the state had gained control of the reservoir site, surveys and foundation tests were made for the dam. The contract was let and construction begun in August, 1907; it was completed in November, 1908. This third dam was washed out in May, 1914 (see Fig. 2).

Description of Site After the Failure

Fig. 1 contains a topographical map of the site as surveyed after the failure of May 25, 1914, and projected thereon, with substantial accuracy, is the official plan of the dam and the outlet culvert, the spillway being shown as it exists. At the northerly end of the site is the steep face, about 10 ft. high, of a bed of basaltic lava the base of which has practically the same elevation as the top of the dam. Extending northerly under the lava is a bed of gravel about 25 ft. thick underlain by sedimentary rock, of which an outcrop, probably uncovered during the failure, appears just below the dam on the northerly edge of the river, and of which another outcrop, known before the construction of the dam, is exposed on the southerly bank of the river one-quarter mile below.

*625 Newhouse Building, Salt Lake City, Utah.

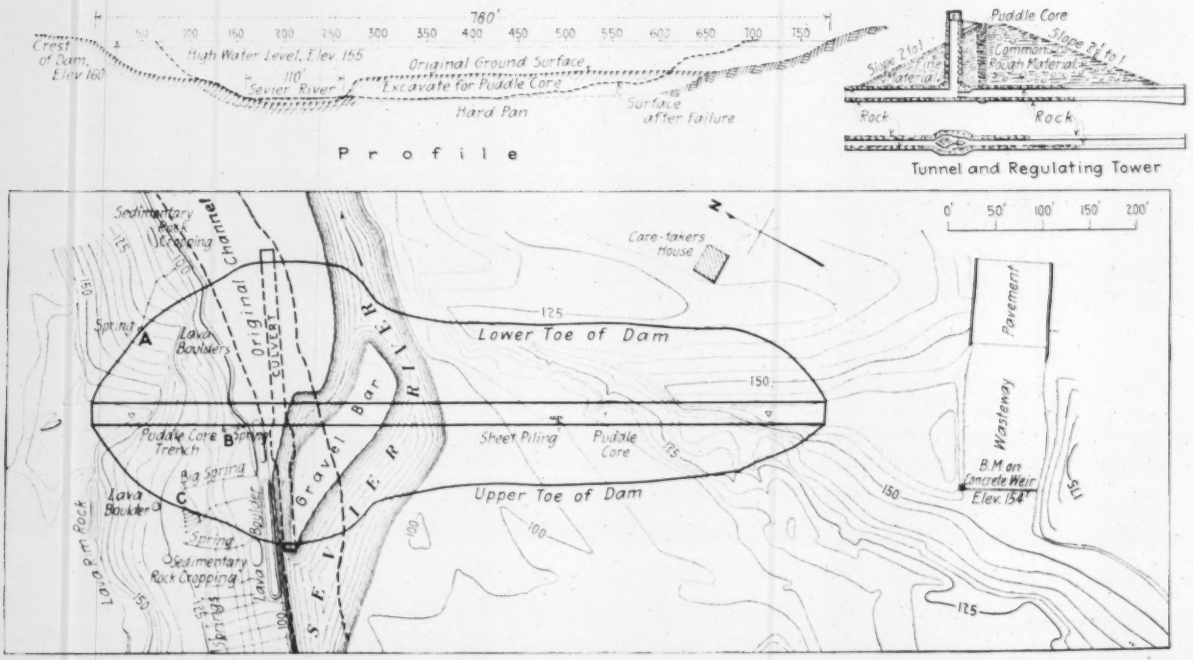


FIG. 1. MAP OF HATCHTOWN DAM SITE AFTER FAILURE AND DETAILS OF DAM

The talus extending southerly from the base of the lava to some distance under the river, on which a large part of the dam was founded, is a heterogeneous mixture of black clay, sand and gravel, on the surface of and embedded in which are many large lava boulders occurring singly and in extensive clusters (Fig. 3). From the gravel underlying the lava and from the bedding planes of the sedimentary rock at the northerly end of the site, about 5 sec.-ft. of water is discharged by several small streams or springs, but chiefly as percolation or seepage. Of the total flow thus discharged probably one-half comes to the surface under the base of the dam. The assumption is that the greater part of this flow consists of seepage from the irrigation of bench lands north and west of the site. On account of this water, which spreads itself quite generally over the talus, the material of the latter, as it appeared three weeks after the failure, was in a saturated condition, being particularly soft where pockets of clayey sand and small gravel occurred. On the southerly side of the river no lava appeared in place, the material in view and on which the dam was based being entirely different from that in the above-mentioned talus and, in general, part of an extensive stratified deposit of sand and gravel.

Comparisons of the topography and profiles (Fig. 1) of the site made before and after the failure indicate that northerly of the river and upstream of the axis the erosion due to the failure and the ensuing flood was in general limited to the 1 or 2 ft. of original soil not removed in clearing the site. Below the axis and especially near the downstream toe the erosion was greater. Fig. 2 shows the eroded face of a bed of boulders partly embedded in the talus material and extending from a short distance below the axis to the downstream toe of the dam. Southerly of the river the surface of a great part of the base is in general from 15 to 20 ft. below the original surface. Since, however, the outlet culvert and nearly all of the sheetpiling, to be described hereinafter, disappeared from the site, it is probable that during the flood the base under the dam southerly of the river and in the river bed also was eroded to greater depths than are now indicated and was subsequently filled to the present levels as the flood subsided.

From the data available regarding foundation tests at the dam site it is difficult to determine whether the "hardpan," stated in the engineer's reports of the tests to have been found at the site, was a bed of clay or the sedimentary rock previously referred to, except perhaps on the northerly side of the river, where the rock was probably reached. The records, however, support the conclusion that on the southerly side mud and quicksand were encountered near the level of the river in every test hole sunk, and that satisfactory investigation of conditions below the quicksand was not made.

FIG. 2. LOOKING NORTHERLY ON AXIS OF DAM AFTER FAILURE

General Details of Dam Construction

Except when otherwise ascertained and noted the design (Fig. 1) and specifications of the dam filed in the office of the State Engineer must be assumed to represent the structure as it was built. According to the data thus available the main structure, of about 126,000-cu.yd. contents, was an earthen embankment of maximum height 63 ft., crest length 780 ft. and contained a puddle corewall from end to end. The slope of the downstream face was 2½ to 1. The slope of the upstream face for a vertical distance of about 10 ft. below the crest was 1½ to 1, with the remainder 2 to 1. This latter face was paved to a thickness of from 1 to 2 ft. with roughly cubical lava rock, generally of such dimensions and weight as one man could handle. The crest width of the dam overall was designed to be 20 ft., but the width of the remnant still standing is less than 12 ft. About 200 ft. beyond the end of the dam and through the gravel and sand ridge forming the southerly abutment was cut the spillway channel, of 150-ft. width as originally designed, but as built, of 80-ft. width, 5- or 6-ft. depth below the top of the dam and of 160-ft. length, to discharge on the paved slope of the downstream side of the ridge. For regulation purposes the spillway was provided with movable weir boards supported by timber frames.

With the given freeboard of 5 ft. below the crest the capacity of the reservoir back of the dam was 13,000

acre-ft., but with a freeboard of 10 ft. the capacity would be reduced to 11,000 acre-ft.

Located in the river bed about 200 ft. from the northerly end and under the deepest part of the dam were the outlet works, consisting of a single rubble-masonry arched culvert 320 ft. long and 5 ft. wide by 8 ft. high, inside, provided with a reinforced-concrete double-compartment gate chamber opening into and supporting a rubble-masonry gate tower situated upstream of the puddled corewall and 35 ft. from the axis of the dam. The arch of the culvert was semicircular, and 3 ft. thick at the crown; the side walls were 4 ft. thick at the level of the floor, under which the masonry was 3 ft. thick. The culvert and tower were lined with cement mortar 2 in. thick. No cutoff collars, as generally understood, were provided for the culvert.

To regulate the discharge through the culvert one 4x11½ ft. cast-iron gate connected with raising devices

FIG. 3. TALUS MATERIAL UNDER BASE OF DAM
Broken face of dam in background

in the tower was installed in each compartment of the gate chamber. No provision was made for repairing the gates or the upstream part of the chamber when the former were closed and the water stood in the reservoir.

The trench for the outlet culvert was excavated in the talus material in the bed of the river, and as shown in Fig. 1 was for about 130 ft. of its length immediately alongside the remnant of the old culvert before mentioned.

Fig. 4 is a typical view on the northerly side of the old culvert, as it appeared after the failure of the dam. The boulder on which the pick in the view rests and the larger boulder at the right are immediately under the masonry and are in their original places as deposited in the talus, although the clay, sand and gravel originally surrounding them have been eroded. The elevation of the base of the old culvert is about 2 ft. above the normal surface of the water in the river, the annual rise and fall of which is seldom more than 3 ft. The rocks forming the foundation course of the culvert masonry were evidently not tied to the "hardpan" but were laid directly on the talus material occurring in the bottom of the trench. In preparation for the next course of masonry concrete was probably shoveled into the interstices between and on top of the rocks. Whether the trench was adequately drained before the concrete was placed, could not be directly determined. Some evidence on this question is furnished by a large piece of masonry estimated to weigh 18 tons and apparently comprising the floor and base of part of the gate chamber, which was carried about 2,000 ft. below the site by the rush of water following the failure of the dam. The rocks of the foundation course as here shown were partly embedded in a hard concrete, but the projecting parts of the rocks and the interstices between them were entirely open and clean, thus indicating that the rocks as laid were partly embedded in some comparatively soft material and not tied to the sedimentary rock underlying the site.

To prevent the flow of water between the old and new culverts a masonry wall of perhaps 2 or 3 ft. in thickness is reported to have been built between the two structures at each end of the gate chamber. But since the base of the old culvert was 8 ft. or more above that of the new one, and since these walls did not rise much if any above the spring of the arch of the new culvert, they naturally terminated in the talus material on which the old culvert was based. Except these walls nothing in the nature of a cutoff collar was constructed in connection with the culvert. Backfilling of the culvert trench is not mentioned in the specifications, and there is no record as to how this portion of the work was performed.

Construction of Puddle Corewall

The center line of the puddle corewall was located about 20 ft. upstream from and parallel to the axis of the dam. The trench for the corewall was specified to be 15 ft. wide through the length of the dam and excavated to "hardpan" or such material as was satisfactory to the engineer. As indicated in Fig. 1, it was 8 ft. wide at its base, but as exposed at its northerly end (Fig. 5) it was 20 ft. wide. The specifications required the core to be built up in 6-in. layers of fine material containing 18% of clay and not over 10% of stones having maximum dimensions of less than 3 in. After deposition the material in each layer was to be mixed by harrowing with a weighted disk harrow, sprinkled and then tamped by driving horses, cattle or sheep over the layer until the hoofs of the animals should penetrate the surface not more than 1½ in. Subsequent to this tamping and in preparation for the next layer the surface was to receive a light harrowing.

The records of the work do not indicate that any difficulty was encountered northerly of the river in excavating the core trench to a base deemed suitable by the engineer in charge. This part of the base, however, was to the talus material previously described, as is shown in Fig. 3 and in the foreground of Fig. 5. At the intersection of the corewall with the old culvert masonry the latter was removed so that at this point the base of the corewall rested directly on the talus material underlying the

old culvert. On the northerly side of the river and within about 10 ft. of the side of the new culvert the corewall was based on the talus material, and consequently, in connection with other described conditions, it is fairly inferable that the new culvert was based on the same material.

The available records give no information regarding the character of the material encountered in excavating that part of the core trench between the culvert and the original southerly bank of the river. Southward from the southerly edge of the river, however, the trench was apparently carried down through the gravel and sand formation to about the level of the river's surface, when mud and quicksand were encountered, preventing further excavation of the trench by the means at hand. It was then decided to drive timber sheetpiling from the bottom of the trench to "hardpan," which was believed to occur at a depth of 4 or 5 ft. below the level of the river's surface. The sheetpiling consisted of a double row of 2x6-in. plank driven by a 20-lb. maul, without staggering the vertical joints. The piling extended southerly from the river's original edge at least 220 ft., where a small portion of the row is still in place with the tops of the piles about 7 ft. above the surface of the river. The balance of the sheetpiling driven southerly of the river has disappeared. Further interesting information regarding the driving of the sheetpiling is contained in the following quotation from a letter written during the work by the superintending engineer.

> We have driven a double row of about 40 ft. of sheetpiling 4 ft. long after taking down the water about 18 in. in the core trench. We have gotten out of the black clay almost entirely and have instead genuine quicksand. We strike something hard in the bottom which the piles will not penetrate though a bar can be driven down with a sledge hammer seemingly without limit.

From this statement it appears probable that the sheetpiling referred to did not reach the so-called "hardpan."

The corewall remaining northerly of the river (Fig. 5) appeared to consist of backfilled talus material. It lacked firmness, cohesiveness and homogeneity. The remnant of the corewall southerly of the river, about 260 ft. from the southerly end of the dam, consisted of clayey sand, which when dry would break up into hard lumps, but when wet quickly disintegrated. This material was similar to that occurring adjacent to the core trench.

Method of Draining the Foundation

Neither the plans nor the specifications of the dam contain any information as to the work performed in disposing of the previously described flow of water which percolated quite generally throughout the talus material, forming the base of the dam northerly of the river. But it is understood that two and perhaps more drainage ditches running in general transversely to the axis of the dam were excavated in the talus material, and that the water thus collected was conducted in 4-in. tiling across the core trench to the downsteam toe of the dam. Appearances at the site after the failure indicated that the ditches were not excavated to impervious material or were not deep enough to intercept more than a small proportion of the total flow, although they probably received such seepage as occurred in relatively large streams and near the surface. Of these drainage works the remnants of two lines of tiling are all that were seen at the site. The broken end of one of these lines of tiling discharging a stream of water (spring *B*, Fig. 1) appears near the downstream side of the core trench (Fig. 5). In the downstream toe of the remnant of the dam northerly of the river were two caves formed long before the failure and resulting from the escape of spring *A* (Fig. 1) from its tiling. Except the spring *C* (Fig. 1) the springs *A* and *B* were the largest instances of concentrated flow to be seen at the site subsequent to the failure. The spring *C* has a discharge about equal to that of the springs *A* and *B* combined. It issues between large boulders partly embedded in the talus material and is located approximately 20 ft. higher than the river and 15 ft. downstream from the upstream toe of the dam, where the depth of embankment directly over the spring was about 8 ft. The occurrence of this spring may be due to an increase, subsequent to the construction of the dam, in the total discharge from under the lava, or merely to a subsequent local concentration of part of the flow percolating generally through the talus. In any case its disposal was not provided for when the drainage works were built.

FIG. 4. TALUS MATERIAL UNDER BASE OF OLD CULVERT MASONRY

A line of tiling extending from the corewall to the downstream toe was also laid on each side of the outlet culvert.

Method of Making Embankment

It was specified that upstream of the corewall fine material should be used for the embankment, to be built up in longitudinal layers having a uniform thickness of not over 6 in., consolidated by a roller of 1-ton weight per foot of length and prepared for the succeeding layers by light harrowing with a disk harrow. Wetting or sprinkling the layers is not mentioned in the specifications of this part of the work. The downstream portion of the dam was to be built of coarser materials deposited in 1-ft. layers and rolled as specified for the upstream fill.

The embankment upstream of the corewall could not be examined, as it was practically all carried away at the time of the failure. The material remaining in the downstream portion of the embankment was a loamy sand lacking compactness but otherwise suitable for the purpose.

Upstream of the corewall, about 120 ft. in length of the remnant of the old culvert and also the boulders and a piece of the broken-down old arch, which in the course of the work had been piled alongside and on top of the old

masonry, were buried under the dam. The occurrence of a piece of old arch in the midst of the boulders indicates that the positions of the latter were practically the same after the failure as before, although the finer material in their neighborhood had been eroded.

By an earthen dike built across the Sevier River above the site the stream was diverted from its channel and carried to below the site on the northerly side of the remnant of the old culvert, to the lower end of which was added a bank of rock, gravel and earth extending downstream beyond the toe of the dam. Later the stream was carried through the new culvert.

The excavation of the outlet culvert trench and, southerly of the river, the corewall trench and the building of the main embankment were started next and carried on practically at the same time.

In obtaining material for the dam, borrow pits were opened on the southerly part of the site, one pit being located immediately adjacent to the upstream toe and at an elevation of from 30 to 40 ft. below the crest of the dam.

The records clearly show that at times during latter November and December, 1907, the ground at the site was frozen hard and that the superintending engineer desired to protect the culvert masonry from frost, but was apparently unable to do so satisfactorily until practically all of the masonry, except the gate-chamber arches, had been placed, the completion of these arches being in fact postponed on account of the cold weather.

Work on the embankment did not cease until about Dec. 1, although the surfaces of the fill as made each day and when not frozen hard were undoubtedly subjected to the deteriorating effects of alternate freezing and thawing due to the cold nights and the succeeding comparatively warm days usual at the site during latter October and November.

FIG. 5. LOOKING NORTHERLY ON LINE OF CORE TRENCH AFTER FAILURE OF HATCHTOWN DAM

In addition the records show that considerable friction existed between the contractor and the superintending engineer, who during October, November and December of 1907 claimed in substance that the contractor was unwilling to perform the work according to specifications or instructions and not equipped either financially or physically to carry on the construction properly. After the resumption of work in April, 1908, there were apparently no further serious differences between the contractor and the engineer.

Operation and Failure of the Dam

In ordinary seasons the storage of water in the reservoir back of the dam would begin between Nov. 1 and Mar. 1 and continue until the following June 1, the greater part of the water being received during the flood season of April, May and June. About June 1 withdrawal of water for irrigation would begin, and by Sept. 30 the reservoir would be practically empty. Thus in general the dam would have to resist the forces due to full depth of water in the reservoir for a few weeks only each year.

In the spring of 1909, the first year of operation of the reservoir, the mechanism of one or both of the outlet gates became disarranged, and the superintending engineer blew them out from the downstream side with dynamite, but reported that slight damage had been done to the gate seats or the gate chamber.

In the spring of 1910, after the reservoir had been filled, seepage and consequent sloughing of about 200 ft. in length of the downstream toe of the dam occurred at an elevation of from 25 to 35 ft. below the crest and southerly of the river. The sloughing was stopped by placing on the toe a heavy layer of rock. The records, however, do not state whether this stopped the seepage.

The records show that during the winter of 1910-1911 a large part of the paving on the upstream face of the dam was relaid and that repair work was performed on the gates, the assistant engineer in charge reporting that the concrete of the bearings of the gates was of poor quality.

On Apr. 15, 1914, the depth of the water in the reservoir at the dam was 35 ft., corresponding to a storage capacity of 4,900 acre-ft. At this depth only about 275 ft. in length of the upstream face of the dam would have been flooded. May 5 the depth was 41 ft.; on the tenth, 47 ft.; on the twenty-second it was 52 ft. and so remained until the twenty-fifth. At 2 p.m. on this latter date, according to the watchman, a slight seepage that had been previously noticed near the bottom and on the southerly side of the downstream end of the outlet culvert increased to a solid stream of muddy water, described as being of the size of a stove pipe, and probably issuing under some pressure. In about two hours this stream began to increase, and at 8 p.m. great wedges of earth began to slip from the face of the dam over the culvert.

In this manner the embankment was quickly cut back to within about 60 ft. of the crest, when a section of the dam, 30 ft. long on the axis, moved bodily forward, followed by a wall of water 52 ft. in depth. A few minutes later the gate tower fell. The watchman estimated that 75% of the 12,000 acre-ft. in the reservoir escaped during the first hour after the break and that by 11 p.m. the reservoir was practically empty.

FIG. 6. UPSTREAM SIDE OF DAM AFTER FAILURE, SHOWING EXPOSED CULVERT END

FIG. 7. DOWNSTREAM SIDE OF DAM AFTER FAILURE SHOWING EXPOSED CULVERT END

The failure of the dam was probably the result of a combination of the described conditions, part of which were contributory in their effect, but some of which were so related that a radical betterment of one or two would probably have prevented the inception of the break and thus saved the structure.

Probable Causes of the Failure

(1) The immediate cause of the failure was, without doubt, the lack of cutoff collars on the culvert.

(2) Of first importance as contributory causes were the defective foundations of the culvert and the corewall, the perviousness of the latter and the failure to clear the site of the remnant of the old culvert and the near-by accumulation of boulders.

(3) An increase in the tendency of the water to creep along the culvert probably resulted from the manner and order of building that part of the dam adjacent to the culvert and based on the river bed, and from the weather conditions prevailing during the intermediate stages of the work.

(4) The excavation in the stratified sand and gravel underlying the southerly part of the site, of a borrow pit parallel and near to the upstream toe at a deep part of the dam, would furnish the water a relatively easy path from the reservoir to the body of the dam, thus contributing to the adverse conditions already existing along the culvert.

(5) The uncontrolled part of the water percolating so generally under the northerly base of the dam would tend to saturate the embankment adjoining the culvert, thus further reducing the resistance to creep of water along the latter. This condition, however, is believed to have been only remotely contributory to the failure, since the pressure of the percolating water appeared insufficient to cause the break. This conclusion applies to the spring *C* also, regardless of when it developed, as its low pressure, combined with its issuance near the upstream toe of the dam, would exclude it as one of the primary causes of the failure.

General supervision of the design and construction of the dam was under Caleb Tanner, then State Engineer, who appointed Joseph Jenson and W. W. McLaughlin, of Logan, Utah, engineers on design. This firm, as shown by the records, prepared the plans and specifications of the dam. With the title of superintending engineer, Joseph Jenson of the above firm had responsible charge of the work in the field through the entire period of construction. The contractor for the work was D. W. Brinton, of Draper, Utah.

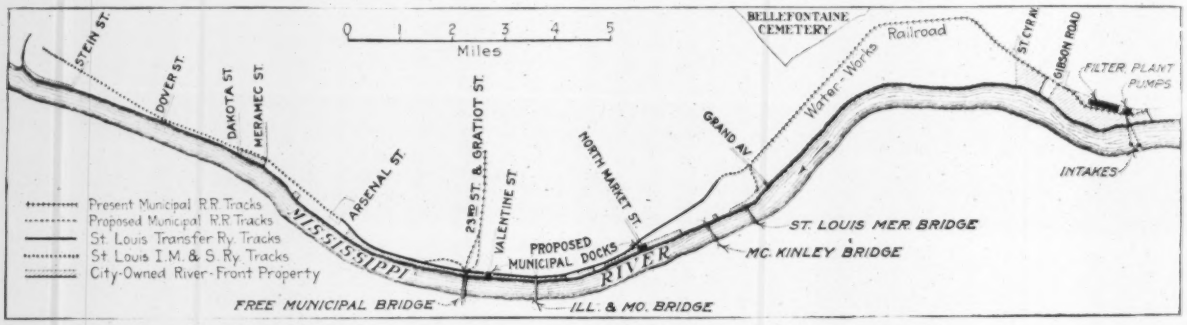
A Private Italian Water-Power Enterprise has been under construction for the past two years near the town of Lucca, in Tuscany. This project, briefly described in "Mine and Quarry" for January, consists of an aqueduct 19,700 ft. in length, which will receive water from the Serchio River. The aqueduct is mostly tunnel, there being two tunnels, one 13,100 ft. long, the other 3,300 ft. in length. The longer tunnel has a portal near Castelnuovo di Garfagnana, while the shorter tunnel is near Gallicano. The Castelnuovo tunnel is designed to be 8x7 ft. in cross-section inside the concrete lining, but the variety of soil encountered caused the section to be changed at a number of points, to facilitate driving and timbering. The soil ranges from soft fire clay to limestone. There is also considerable clayey schist. A bottom heading (6½x6½ ft.) is driven, and the roof is taken down and the tunnel enlarged to full section. One of the headings ran into soft material and required timbering. Harder material was encountered in the other heading of the Castelnuovo. Drilling is done by two Sullivan piston drills, mounted on a heading bar, which put in a round of eight holes to an average depth of 5 ft. In harder rock additional holes were drilled. The order of drilling was as follows: First, the two back holes, looking up, from over the bar; next, the center hole; then the two rib holes, looking up slightly; then the drills were swung under the bar and the three lifters were put in, looking down slightly. Two benzine locomotives are used to haul the muck out of the tunnel. On the shorter tunnel the bottom-heading method is employed also, although a top heading was driven for the first 100 ft. Air is supplied to the drills in the longer tunnel by two cross-compound air compressors located one at each portal. The promoter is the Societa Ligure Toscana di Elettricita, Ltd., of Leghorn. Mario Axerio, of Turin, is engineer. The contractor is Manotti Falzoni, of Milan. The work should be completed by July, 1916.

St. Louis Municipal Railway and Water Front

Unique among cities of the United States through ownership of a large part of its water front, St. Louis is already well started on a project (see accompanying map) which will make it owner of a railroad extending along more than two-thirds of its Mississippi River water frontage, together with a branch inland, the whole totaling about 18 mi. in length. Two municipal docks are proposed, as shown on the map and as was stated in *Engineering News* of Dec. 2, 1915, p. 1101.

The city's first venture in railway ownership was the Water-Works Railroad, 7½ mi. in length. This was completed about 1891, to aid in constructing the Chain of Rocks pumping station, and now gives access also to the large filtration plant put in use in 1915.

A franchise held by the St. Louis Transfer Railway Co. and extending from about the southern end of the city's line to a point about 2 mi. south of the municipal bridge expired. This company is a subsidiary of the St. Louis Terminal Railway Co., and it happened that at the same time, in order to enlarge its yard facilities, the Terminal Co. was applying for permits to cross certain public streets passing through ground which it already owned.



MUNICIPAL RAILWAY AND WATER-FRONT PROPERTY, ST. LOUIS, MO.

After negotiation a temporary permit was issued for these crossings, with the understanding that it must be confirmed by an ordinance and that among the considerations in this ordinance the city would receive the present trackage of the Transfer Railway Co., and the Terminal Co. would also construct a connection between the new municipal bridge and the Merchants Elevated Tracks, which run into the Union Station. This would give the city over 6 mi. of additional trackage, most of which would be in a line extending along about two-thirds of the water front and part of which consists of double tracks.

The city also has the right-of-way and is preparing plans for a line westward from the municipal bridge to meet the Western trunk lines at about 23rd St.

The franchise of the St. Louis Transfer Railway Co. expired about Dec. 15, 1915. The Department of Public Utilities has prepared an ordinance to make effective the agreement described above. All of this work is under James A. Hooke, Director of Public Utilities.

The new docks are to have reinforced-concrete wharf structures, with steel frame warehouses and full mechanical equipment. The first unit, now authorized, will be 300 ft. long out of a total intended length of 1,200 ft. The estimated cost of the work is $285,000.

Tar Coating for Concrete Pipe Under High Velocity*

By Charles H. Paul†

The regulating outlets through the Arrowrock dam are molded in 1:2 portland cement mortar, except for the 28 ft. next the upstream face of the dam, which is lined with semisteel pipe. These outlets are 4 ft. 4 in. in diameter and are subjected to velocities of 60 ft. per sec. or higher. Wood-pipe staves were used for lagging the forms, and great care was used to secure a smooth and straight surface. The upper outlets are 70 ft. in length and the lower ones 130 ft. long. After the removal of the forms they were trimmed, scraped and washed with grout in the usual way, and then they were painted with two coats of water-gas tar and two coats of coal tar—the same combination that has been used for several years for painting metal that is to be subjected to the action of water.

The idea at Arrowrock was not so much that of waterproofing as to fill all the minute voids in the surface of the outlets, so as to prevent if possible the erosion caused by the formation of vacuum in small voids or pockets. Preliminary experiments had indicated that the water-gas tar would penetrate the concrete, and that the coal tar would follow it in and bind to the concrete much better than if it were applied direct, thereby completely filling the voids at and near the surface and coating the concrete with a slick and fairly durable finish, cheap and easily applied.

Behavior of Pipe in Service

This coating was applied to all outlets during the fall of 1914. Five of the lower outlets and six of the upper outlets were in service during the irrigation season of 1915, some of them working for four and one-half months under heads varying up to a maximum of 100 ft., with corresponding velocities up to 64 ft. per sec.

All of them passed through a fairly normal winter and summer. The one that was used most was operated nearly all the time with its valve only partly open, which is a less favorable condition than with the valve wide open. After this service careful inspection was made and the tar coating was found to be in excellent condition. In a few small spots the outer coating had come off, but even there the concrete surface was well filled with the tar and was still well protected apparently against water

*From "Reclamation Record," January, 1916.
†Recently Construction Engineer, United States Reclamation Service; now with Miami Conservancy District, Dayton, Ohio.

action. It is the opinion of the men familiar with all conditions that the use of this tar coating gave thoroughly satisfactory results.

Precautions in Using Tar

In obtaining material for work of this kind it is very important that the water-gas tar be of very thin consistency. The manufacturers can supply it that way if it is made plain to them that this thin consistency is important; otherwise it is likely to come too thick. The manufacturers can also supply an oil for thinning this water-gas tar, and it is advisable to have some of this oil on the job in case it is needed; but it is believed that the success of the whole operation depends upon the very thin, water-like consistency of the water-gas tar. Both the water-gas tar and the coal tar should be refined.

It is necessary to have the concrete thoroughly dry at the time the first coat is applied. The water-gas tar may be applied without heating, and the second coat may follow the first immediately. The first coat of coal tar may be applied as soon as the water-gas tar has soaked in a little, but the second coat of coal tar should not be applied until after the first coat has set. Both coats of coal tar should be applied hot and brushed out as thin as possible, as a thick coating is much more likely to peel and run.

Approximate Costs of Applying Tar

The approximate cost of painting 1,000 sq.ft. of surface in a 52-in. diameter outlet with two coats of water gas and two of coal tar as described herein was as follows:

Labor, 4½ days at $2.50	$11.25
Material:	
12 gal. water-gas tar at 16c.	1.92
15 gal. coal tar at 16c.	2.40
Brushes and miscellaneous	1.43
	$17.00

Cost per sq.ft., say 1¾c.

The refined water-gas tar and refined coal tar were purchased from the Barrett Manufacturing Co. at $4 per bbl. of 50 gal. f.o.b. Chicago. Freight to Boise was about $4 per bbl.

How Co-operative Courses Train Young Engineers

By H. A. Stringfellow*

The writer has read with interest and gratification, the excellent, though all too brief, article by Prof. F. E. Ayer, on "Coöperative Technical Schools," which appeared in the Dec. 2 issue of *Engineering News*. It is aimed here to supplement that article with a paragraph on young engineers as real producers and to give the experience of another school employing the coöperative system in civil-engineering training.

Limited Value of Young Graduate

To say that the graduate of the purely theoretical school is poorly prepared for actual engineering work is to put the case mildly. He is, in the majority of cases, not prepared at all, and in fact has to live down much of his education and training before becoming of value to his employer. He has an education along technical lines that would qualify him ultimately to occupy a high position in the engineering world; but because of his inability to apply this education early he is unfitted to hold efficiently the subordinate positions he must occupy in order to acquire experience. He has been taught to think in large numbers, to look upon a structure as a whole and to overlook the detail operations necessary to its creation. He has slight idea of the value of details, absolutely no knowledge of costs, materials or methods, and no experience to back up his technical training; he is almost valueless to his employer at the start. When put at routine work in the drafting room, where he almost invariably lands upon graduation, he becomes discontented and discouraged because things are so different from what he had pictured them. In many cases he is not even told the use to which his work is to be put. The result is that he becomes of still less value to his employer.

It is the aim of the coöperative system to avoid this condition—this overtraining in theory and lack of training in practice; this unbalanced condition that throws upon his employer the burden and expense of training the technical graduate. It is the object of the coöperative system to turn out men who are producers, small ones to be sure, but who from the start are of value to those who employ them.

Many engineers know in general of the coöperative system, but few have any definite ideas as to how it is operated and its results as regards the students working under it. Such information can best be given by a concrete example.

Coöperative Courses at Rochester

At the Mechanics Institute, Rochester, N. Y., the theoretical work, that is, the work of the classroom and laboratory, is arranged to a great extent upon the same general lines as those followed in engineering schools of the purely theoretical type—with this exception: No courses are given simply for their value in mental or manual training. It must be shown that a certain course has a practical educational value before it is admitted.

Critics of this system will say that it is too intensely practical. Our reply to this is that engineering work is, in its very nature, intensely practical and that such courses have a greater value in mental training because of the fact that the student realizes their value and pursues them with increased interest. As an example we can take the courses in mathematics.

These courses are not given as an end in themselves, but simply as tools to be used in actual work. The problems given are not the vague, pointless propositions encountered in the average text, but bear directly upon actual work. As the student learns the subject, he is taught the practical application of each new theorem or method that he encounters, both by lectures in the classroom and problems to be solved. The result is that when he comes up for his pure-engineering subjects he has a set of tools with the use and construction of which he is familiar.

For work in physics the same idea is followed. In English his work is not confined to themes that simply give him general training in self-expression; he is taught the precise language of specifications, the correct manner of writing business letters, how to prepare a report on a certain engineering project, how to write a technical article.

But it is in the handling of the pure engineering subjects that the greatest value of the coöperative system is to be felt, and the writer will attempt to show the result of the system upon the work of the students.

*Head, Department of Civil Engineering, Mechanics Institute, Rochester, N. Y.